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*J. B. Johnson*

*Dean of the College of Engineering  
of the University of Wisconsin  
1899-1902*

# JOHNSON'S MATERIALS OF CONSTRUCTION

REWRITTEN

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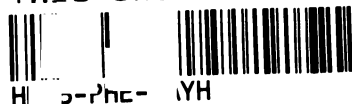
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## PREFACE

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SOME twenty years ago the late Dean J. B. Johnson wrote the following pertinent statement in the preface of the first edition of *The Materials of Construction*:

"The rational designing of any kind of construction involves a knowledge of:

"The external forces to be resisted, transformed, or transmitted;

"The internal stresses resulting therefrom;

"The mechanical properties of the materials to be employed to accomplish the objects sought.

"Of these three coordinate departments of knowledge the first two are founded on the sciences of mathematics and applied mechanics. The last one, however, does not rest on any deductive science, as this information can only be gained by patient, expensive, and competent research. For this reason the third essential named above has not kept pace with the other two kinds of engineering science; but, on the other hand, it furnishes very much greater rewards to the skilled investigator.

"During the past twenty-five years the number of such investigators has increased from a scattering few to hundreds and even thousands, and these are now found in all enlightened nations. The results of their original studies and experiments are pouring in upon us from all countries, in many languages; and no practising engineer can hope to even scan, much less to appropriate and assimilate, more than a very small part of this vast wealth of experimental knowledge."

The belief that it was essential that students and engineers should have a broad knowledge of this subject led that author to compile his well-known treatise. His good judgment and foresight in so doing were confirmed by extended use of the book both as a text and as a reference for nearly a score of years with little revision. It is obvious, however, to those familiar with the great progress that has been made in recent years in the knowledge of the properties of material, that the work no longer adequately meets the need in this field. The number of investigators and the published data of their researches are now so comprehensive that it is well nigh impossible for even the specialist in the materials of construction to keep

abreast of the advances being made, and necessarily much of the information in the former book has now become obsolete.

Believing that there is a distinct advantage in presenting to the student of this subject a book which will serve both as a text and as a reference, the authors of the re-written work have maintained in a large measure the broad scope of the former treatise. They have aimed to provide the essential information concerning the sources, manufacture or fabrication of the principal materials; to give carefully selected data covering the more important mechanical and physical properties and the influences of various factors upon these properties; to show the causes of defects and variations and how they may be discovered; to furnish an acquaintance with the technique of testing materials; and to present to the student some of the more general uses of the different materials.

In arrangement, the new book differs markedly from the former work, which was somewhat cumbersome in this respect. The division into parts has been discarded and related subject matter has been more closely coordinated than in the previous work as may be seen from the following: In Chapter I is given a rather comprehensive synopsis of the principles of mechanics of materials. Chapters II and III deal with machines and appliances for testing, the technique of testing and the utility of the various tests. Chapters IV to VI consider the characteristics, methods of identification, properties and uses of the more important native woods, also causes of decay and means of preservation. Chapter VII treats of the important stones, their constitution, durability and properties. Chapter VIII covers the manufacture and testing of structural clay products, together with their mechanical properties and uses. Chapters IX to XII deal with the nature, manufacture, methods of testing and properties of the hydraulic cements, the limes and the plasters. Chapters XIII to XV describe very fully methods of making mortar, concrete and concrete products, also the properties and uses of these materials. Chapter XVI provides a brief summary concerning the utility of the principal metals, their ores and the fundamental considerations governing their extraction. Chapters XVII to XIX treat of the reduction of iron from its ores and the subsequent operations of purification and fabrication into final form. Chapters XX and XXI deal with the formation and structure of alloys in general, and the constitution of iron and steel. Chapters XXII to XXIV are devoted to a discussion of the properties and uses of wrought iron, steel, and alloy steels. Chapter XXV takes up the manufacture, molding, constitution and properties of cast iron and malleable cast iron. Chapter XXVI treats of the production, properties and uses of copper, zinc, aluminum, lead, tin, nickel and their alloys. Chapters XXVII to XXIX cover the effects of temperature on metals, the causes and effects of fatigue, and the corrosion and protection of metals. It is believed that the new arrange-

ment will be considered an improvement over the old and that it will make the work more serviceable as a textbook for students and as a reference for engineers.

There have also been some changes in the scope of the new work. The space devoted to the principles of mechanics has been materially reduced by more concise treatment. In general, discussion of the properties of composite members has been omitted. The writers feel justified in this omission since these subjects are now well treated in many textbooks on mechanics, structural design, and reinforced concrete. In the portion devoted to timber, space has been conserved by the omission of the descriptions of trees and by a complete revision and rearrangement of material, thus avoiding the duplications present in the earlier book. The methods of microscopic analysis and magnetic testing of metals have been omitted since it is felt that an adequate treatment of these very important special methods of investigation can not be given in a treatise of this character. On the other hand the new work treats more fully than the former concerning structural clay products, limes and plasters, Portland cement, making and testing of concrete, the methods of manufacture of the metals and the parts made of them, the structure and constitution of metals and alloys, the corrosion and protection of metals, and the decay and preservation of timber.

Great care has been taken to illustrate adequately the revised work with material from the best sources. Old figures have been redrawn and, wherever possible, reduced in size. The large use of diagrams and charts in presenting facts and laws, and the omission of tables wherever possible has featured the present volume as it did the previous book. Dean Johnson well said:

"A law of relationship cannot be perceived from data arranged in a tabular form. When plotted to significant arguments the law not only becomes evident at a glance, but when once impressed on the mind through the sense of sight it cannot well be forgotten. To obtain this lasting benefit, however, the diagram must be intelligently read and understood. The reader is urged, therefore, to give great care to the study of all the diagrams which accompany the text on any subject, for, as a rule, the facts, laws, and conclusions to be drawn from them are not fully expressed in the text. The diagrams must be considered as a part of the text, and they should be read with even greater care than is bestowed on the word-embodied ideas."

In the preparation of the present book, Chapters XVI to XXI have been very largely the work of Mr. James Aston, Metallurgist with A. M. Byers Company; Chapter XXVIII has been written by Professor J. B. Kommers of the Mechanics Department, and Chapter XXIX by Professor O. P. Watts of the Chemical Engineering Department of The University of Wisconsin. The remainder of the book has been the work of Professor



M. O. Withey, also of The University of Wisconsin, to whom great credit is due for the vast amount of painstaking work he has done in the preparation of this treatise. In exercising general oversight as editor, the undersigned has endeavored to adhere to the purpose which inspired the first edition and to produce a work which would be of real service to students and engineers.

Acknowledgment of the many sources of information consulted in the compilation of this volume have been made in the text. The writers are also greatly indebted to Professors E. R. Maurer and R. S. McCaffery of The University of Wisconsin, to Professor D. A. Abrams of Lewis Institute, to Messrs. J. A. Newlin, C. J. Humphrey and C. H. Teesdale of the Forest Products Laboratory for assistance rendered during the preparation of the manuscript.

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F. E. TURNEAURE, *Editor*.

MADISON, September, 1918.

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Then 
$$E = \frac{\frac{P}{A}}{\frac{e}{l}} = \frac{S_t}{\epsilon} \quad \text{or} \quad \frac{S_c}{\epsilon} \quad . . . . . (1)$$

### 5. Longitudinal and Lateral Deformation under Direct Stress.—

When a body is subjected to a direct stress, either tension or compression, it undergoes a certain amount of lateral as well as longitudinal deformation. The ratio of lateral to longitudinal deformation is called Poisson's ratio, denoted by  $\lambda$ . The values of this ratio for some of the more common materials are as follows: \*

Glass.....	0.2451	Brass.....	0.3275
Steel.....	0.2686	Delta-metal.....	0.3399
Copper.....	0.3270	Lead.....	0.4282

**6. Volumetric Deformation.**—If the length ( $l$ ) of a body is increased by  $e l$ , its lateral dimensions are decreased in accordance with Art. 5 and the new volume of a rectangular bar having lateral dimensions of  $b$  and  $d$  would be

$$l(1+\epsilon) \cdot b(1-\epsilon\lambda) \cdot d(1-\epsilon\lambda) = lbd(1+\epsilon-2\epsilon\lambda). \dagger$$

But the original volume was  $lbd$ , hence the change of volume is  $lbd(1-2\lambda)\epsilon$ , and the relative change is  $lbd(1-2\lambda)\epsilon$ , divided by the original volume, or  $(1-2\lambda)\epsilon$ .

If we now apply an equal direct tension in the direction of  $b$ , we would increase this dimension by  $\epsilon b$ , and the volume by  $lbd(1-2\lambda)\epsilon$  as before. A similar result is produced by a tensile force in the direction of  $d$ ; hence, for a direct tensile force in all three directions, the volume will be increased by  $3(1-2\lambda)\epsilon$  times its original volume, and each dimension by  $(1-2\lambda)\epsilon$  times its original value. For a compressive force in all directions the volume will be diminished in the same ratio.

The volumetric modulus of elasticity for equal stresses in all directions will be equal to the unit stress divided by the relative strain

$$3(1-2\lambda)\epsilon \quad \text{or, if } E_v = \text{volumetric modulus, } E_v = \frac{S}{3(1-2\lambda)\epsilon}.$$

But  $\frac{S}{\epsilon}$  is the value of Young's modulus or  $E$ , hence,

$$E_v = \frac{E}{3(1-2\lambda)}. \quad . . . . . (2)$$

If, for example,  $\lambda = \frac{1}{4}$ , then  $E_v = \frac{2}{3}E$ .

\* Taken from Wertheim and given in the Report of the French *Commission des Méthodes d'Essai des Matériaux de Construction*, 1895, Vol. 3, p. 6. For  $\lambda$  for concrete and stone see Art. 521 and 271.

† Omitting terms containing  $\epsilon^2$  and  $\epsilon^3$ , as  $\epsilon$  is a small quantity.

# 7. Shearing Deformation and Shearing Modulus of Elasticity.—

Let  $ABCD$ , Fig. 1, represent a very small element of a body subjected to the shearing stresses  $V$ . The dimensions perpendicular to the plane of the paper may be taken as unity. For equilibrium the shearing stresses  $V$  must be equal on all four faces, the couple formed by the two vertical forces being balanced by that formed by the two horizontal forces. The unit shearing stress will be  $\frac{V}{l} = S_s$ .

Taking a diagonal section on the line  $AC$ , it will be found that the stress on this section will be purely tensile and equal in intensity to the shearing stress. That is,

$S_t = S_s$ . Likewise on the diagonal  $DB$ , the stress is compressive and has an intensity of  $S_c = S_s$ . The element will be deformed into a rhombus, shown in Fig. 2. Assuming the diagonals to retain their original direction each side will be deflected through an angle  $\theta$ , and the total change of each apex of the original figure will be  $2\theta$ . This angular deformation is a measure of the unit shearing deformation, and the unit shearing stress  $S_s$  divided by this relative deformation, is called the *modulus of in shear*. Or,

$$E_s = \frac{S_s}{2\theta} \quad \dots \dots \dots (3)$$

The value of  $\theta$  can be calculated by a consideration of the effect of the tensile and compressive stresses  $S_t$  and  $S_c$ . The tensile stress  $S_t$

will increase the length along the diagonal  $DB$  by the amount  $\frac{S_t l}{\cos 45^\circ E}$ ,

and will shorten the diagonal  $AC$  by the amount  $\frac{S_t l}{\cos 45^\circ E} \lambda$  where  $\lambda =$

Poisson's ratio. The compressive stress  $S_c$  causes a similar effect. The increase of diagonal  $DB$  and total shortening of diagonal  $AC$  will then be equal to  $\frac{S_t l}{\cos 45^\circ E} (1 + \lambda)$ .

From Fig. 2 the angle  $\theta$  (in radians) is practically equal to  $\frac{AE \cos 45^\circ}{AF}$ ; and, substituting from the preceding values, we have

$$\theta = -\frac{S_s(1 + \lambda)}{E} \quad \dots \dots \dots (4)$$

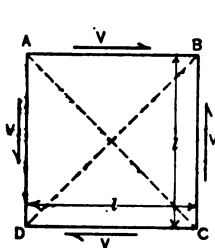


FIG. 1.

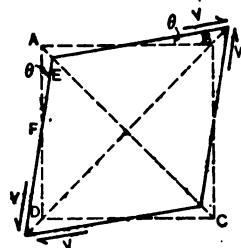


FIG. 2.

The modulus of elasticity in shear is therefore equal to

$$E_s = \frac{S_s}{2\theta} = \frac{S_s E}{2S_s(1+\lambda)} = \frac{E}{2(1+\lambda)} \quad \cdot \cdot \cdot \cdot \cdot \cdot (5)$$

This equation gives the theoretical relation between the shearing modulus and the modulus in direct stress.

In practice, it is difficult to determine experimentally the exact value of  $E_s$  corresponding to the theory herein given, on account of the presence of other stresses than those here considered and the fact that the shearing stresses vary from point to point in a test specimen. However, experimental values correspond approximately with the theoretical values determined by the above equation.

In the case of steel, for example,  $E = 30,000,000$  lb. per square inch,  $\lambda = .27$ , hence from Eq. (5)  $E_s = \frac{30,000,000}{2.54} = 11,800,000$ . Bauschinger found from tests on round bars  $E_s = 13,600,000$ , and from tests on square bars  $E_s = 11,500,000$  lb. per square inch.

**8. Characteristic Behavior of Materials under Stress.**—Materials differ very widely in their behavior under stress. Some of these differences, such as elasticity and plasticity, have already been mentioned. Other common characteristics are indicated by the following terms: ductile, brittle, stiff, flexible, tough, malleable, and hard.

*Ductile* materials are capable of being drawn out without necking down. Wrought iron, soft steel, and copper are ductile metals.

*Brittle* materials have little or no plasticity. Such materials show little deformation beyond the elastic limit, and will therefore fail suddenly and generally without warning. Cast iron, stone, brick, and concrete are materials that are comparatively brittle, but which have, however, a considerable amount of plasticity.

*Stiff* materials have a high modulus of elasticity, that is, a high ratio of stress to deformation. They deform little for a given load. Another sense in which this word is used is to indicate a material with relatively high elastic limit. Thus, a spring made of hard steel will have the same deformation per unit of load as one made of soft steel (the values of  $E$  being the same), but it will carry a greater load than the spring of soft steel, as the elastic limit is higher. It is therefore often called a *stiffer* spring. Technically speaking, the two have the same degree of stiffness.

*Flexible* materials will bend considerably without rupture. They have a low modulus of elasticity, or a low elastic limit and considerable ductility or plasticity beyond that limit. Willow and hickory are flexible woods.

*Tough* materials will withstand heavy shocks or will absorb a large amount of energy. Toughness is dependent upon strength and ductility

or strength and flexibility. Mild steel, wrought iron and hickory are tough materials.

*Malleable* materials can be hammered into thin sheets without rupture. Malleability is dependent upon the ductility and softness of the metal. Copper is very malleable, wrought iron and soft steel are somewhat malleable at ordinary temperatures but may be more easily worked at a red heat.

*Hard* materials offer high resistance to scratching or denting. They are not necessarily of great strength. White cast iron and chrome steel are such metals. Materials which offer high resistance to abrasion are also called hard, although they may not be highly resistant to indentation. Manganese steel is an example of such material.

## MATERIALS UNDER TENSILE STRESS

**9. General Phenomena accompanying Tensile Tests.**—When a body of uniform cross-section is subjected to the action of a tensile force, it is elongated in the direction of this force by a proportionate amount equal to the average force per square inch divided by its modulus of elasticity; thus

$$\frac{\epsilon}{l} = \text{proportionate elongation} = \frac{S}{E},$$

where  $\epsilon$  = total elongation,  $S$  = force or stress per unit area and  $E$  = modulus of Elasticity (Young's modulus). At the same time its lateral dimensions are reduced in accordance with Poisson's ratio, as described in Art. 5. The rate of elongation in the direction of the force, and contraction in its transverse dimensions, continues in strict proportion to the amount of the external force, until the elastic limit is reached, when both the longitudinal elongation and the transverse contraction begin to increase at a more rapid rate, until finally, with the more ductile metals, the condition of perfect plasticity is reached, and the body elongates under a constant force, while the lateral dimensions reduce more and more, until rupture finally occurs.

If the external force or load, in pounds per square inch, be represented by vertical ordinates, and the corresponding elongations be represented by horizontal abscissæ, then the action of the specimen under test may be indicated by what is known as a stress-diagram, the vertical coordinates representing stress, and the horizontal coordinates the corresponding deformations. In Fig. 3 such stress-diagrams are shown for zinc, cast iron, wrought iron, and steel. These lie on the upper side of the horizontal axis. If the same materials were to be subjected to compressive external forces, corresponding stress-diagrams might be drawn in opposite directions, that is to say, downward and to the left, as indicated in Fig. 3, below the horizontal axis.

In Fig. 4 are shown portions of these same tensile diagrams with the deformation scale largely magnified, so as to bring out more clearly the characteristics of the various curves for small deformations. It will be noted that the diagram for zinc is curved almost from the beginning; the diagram for cast iron is straight for only a short distance; the diagrams for wrought iron and steel are straight until the stress has reached 50 to 60 per cent of the ultimate strength. The diagrams for zinc and cast

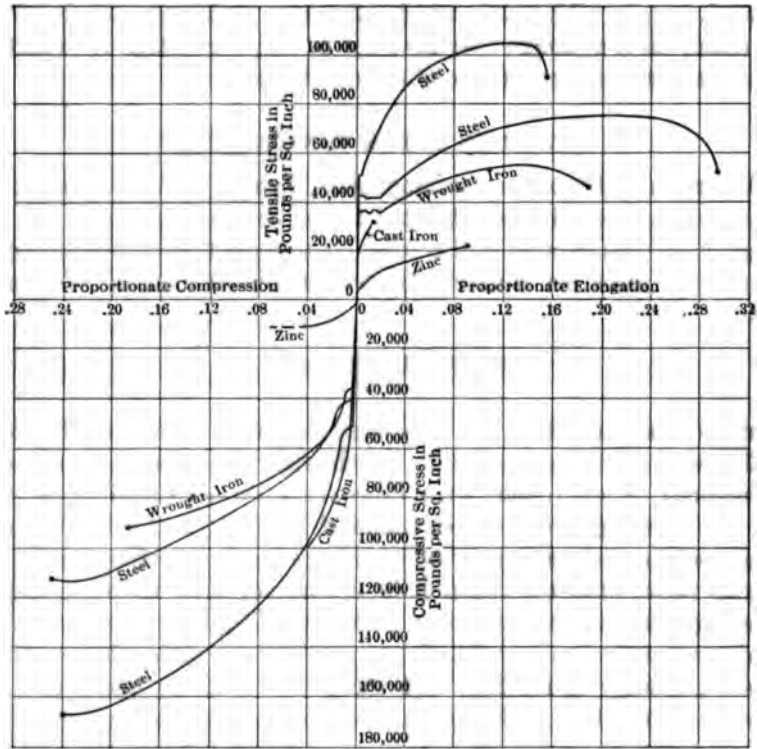


FIG. 3.—Typical Stress-diagrams of Rolled Zinc, Cast Iron, Wrought Iron, and Steel in Tension and Compression.

iron are typical for materials of a non-homogeneous nature. Stone, brick, cement, concrete and some of the brasses and bronzes behave in much the same way, as shown by the diagrams in Art. 272, 300, 516 and Ch. XXVI. Many of the more ductile metals behave in a manner similar to the wrought iron and soft steel; when the point of plasticity is reached (Y.P.), a considerable deformation occurs with little or no increase of load, thus giving a horizontal notch in the curve.

**10. The Significant Results of a Tensile Test.**—There are five significant results of a tensile test, namely:

The elastic limit;  
 The modulus of elasticity;  
 The ultimate strength;  
 The percentage of elongation;  
 The reduction of area of cross-section.

**11. The Elastic Limit.**—In Art. 3, the elastic limit was defined as the limit of unit stress below which the material would fully recover its form upon removal of the load.

Another definition is commonly employed, especially in connection with the study of stress diagrams. Under this definition the elastic limit is the limit of proportionality of stress and deformation; or it is the unit stress on the diagram where the curve departs from a straight line. As a matter of fact, it is found that the two definitions substantially correspond; that is to say, the stress at which the limit of proportionality is reached is practically the same as the limit of stress for complete recovery of form. In connection with the study of materials from the usual static tests, it is more convenient to consider the limit of proportionality as the elastic limit.

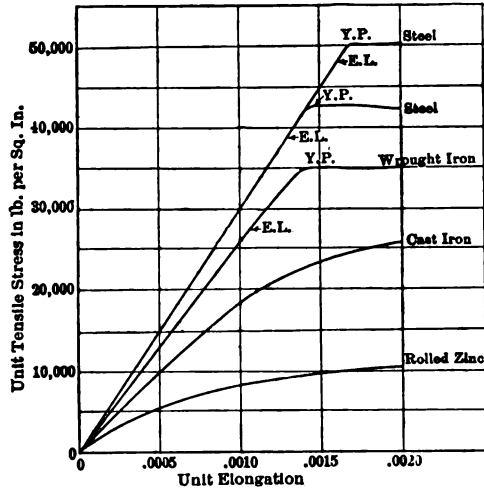


FIG. 4.—Typical Tensile Stress-diagrams for Rolled Zinc, Cast Iron, Wrought Iron and Steel to Enlarged Scale

In the case of such materials as timber, stone, and concrete, the true elastic limit is very low, as these materials will show a small set for very low loads. For most metals, the diagram is sensibly straight for a long distance, and the true elastic limit is relatively high. Even in this case, however, the exact point of departure of the curve from a straight line is difficult to determine, and its location will depend to a considerable extent upon the precision of the observations. It is not, therefore, a point which can be very readily determined. On this account it is customary practice in commercial testing of wrought iron and structural steel to determine the point where the deformation increases rapidly. This point is called the yield point. It is also sometimes called the *apparent elastic limit*. For many purposes it is sufficiently near the elastic limit to be used as such, but in other cases, it is very considerably beyond that limit and should be used with caution.



The original author of this work proposed that, in view of the difficulty of determining the true elastic limit, an *apparent elastic limit* be taken as the point on the stress diagram at which the rate of deformation is 50 greater than it is at the origin. Under this definition, the apparent limit would practically correspond to the yield point in materials such as a point and would give a reasonable value for such materials: iron or hard steel, for which this diagram shows a very gradual cur away from the straight line. Such a criterion has much merit, and accomplish the following results:

1. It would always fix one and the same well-defined point.
2. This point would always correspond to so small a permanent

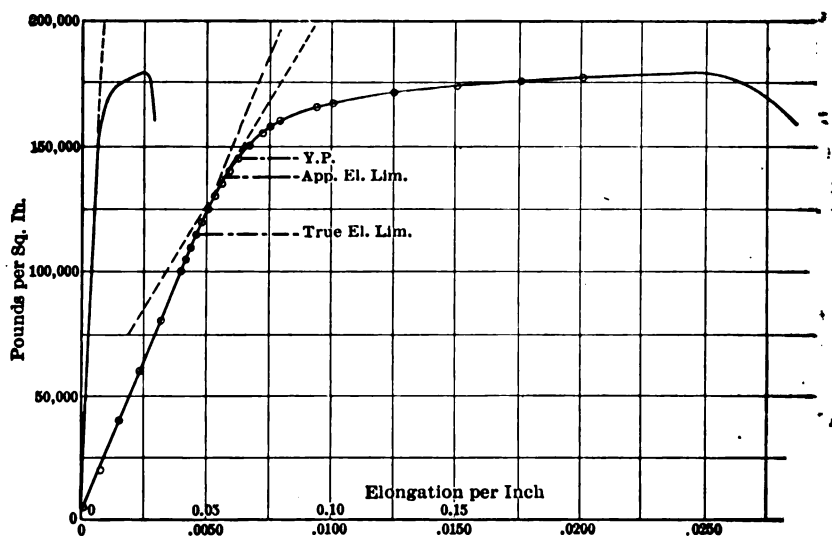


FIG. 5.—Stress-diagram of Hard-drawn Steel Wire. (*Tests of Metals*,

deformation as to be, for many practical purposes, the true el

3. It is equally applicable to all kinds of tests, whether on or on finished members or structures, where deformations of any l be correctly measured.

While the 50 per cent increase in the rate of deformation is arbitrary, it is not large enough to fix a point having an appreci manent set, but it is large enough to fix a well-defined point on 1 diagram.

Fig. 5 illustrates the relation between the apparent elastic limit proposed and the true elastic limit and yield point.

**12. The Modulus of Elasticity.**—The *modulus of elasticity* is fo dividing any stress per square inch below the elastic limit by t

the information. Since the stress-diagram is a straight line to the elastic-limit point, any point in this portion of the diagram is suitable for the determination of the modulus of elasticity. The modulus of elasticity is the tangent of the angle of the stress diagram below the elastic limit forms. Thus when the two coordinates are properly evaluated on the vertical and horizontal scales, respectively.

materials having a curved diagram almost from the beginning of elasticity is not so readily defined or determined. In such cases, it is taken as the slope of the tangent at the beginning of the curve. However, as in the case of concrete, the actual working stress is above the straight portion of the diagram; there is a slight permanent set in such material under working stress. In such case it is desirable to consider the modulus of elasticity as the secant line—see Fig. 11, Chapter XIV—drawn from the origin to the point in the curve corresponding to the unit stress ( $S$ ). The slope of this line is then called the modulus of elasticity—stress  $S$ . This use of the term modulus is especially common in reinforced concrete.

**Ultimate Strength.**—The ultimate strength of a specimen under stress is measured by the maximum load carried, and in the stress-diagram by the true maximum point in that diagram. It is found by dividing the maximum load by the original area. In case of the more plastic metals, the area of the broken specimen is about one-half the original area, so that the ultimate strength of the material is found by dividing the breaking load by the original area. That is to say, the drawing of the metal has nearly doubled its strength per square inch. *ultimate strength," however, always refers to the original strength, and is found by dividing the maximum load by the original section.* The stage of elongation is found by dividing the increase of length by the original length. By original length is meant the length of a certain portion of the specimen which has been reduced to a standard section before testing. A standard length for tensile testing in America and in England is 5 inches, while in Germany it is 10 cm., these standard lengths being practically identical. In the case of a test specimen of the plastic metals may be divided into three parts: (a) That part of the elongation which is uniformly distributed over the section; (b) that part of the elongation which occurs in the necked section which finally breaks. Thus in Fig. 6 are shown three specimens of mild steel, there being three specimens of one set were originally of the length indi-

cated by the untested specimen which stands on the left side of each group. The specimen next adjoining it on the right has been stretched to the limit of the elongation indicated in (a) above, or until there is an indication of a local reduction of area. The right-hand specimen in each group shows the local elongation and reduction, but the specimen has been removed from the testing-machine before rupture occurred. The middle specimen of each group has been tested to the ultimate strength of the material, since, when the specimen begins to reduce locally, the ultimate strength has been passed, and the stress diagram begins to fall, or it is developed under a diminishing load.

By the amount, therefore, that the right-hand specimen in each of these groups is longer than the middle specimen of the group, by so much has

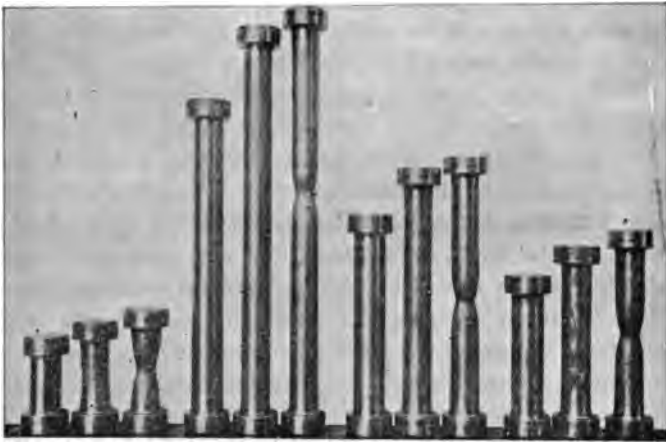


FIG. 6.—Showing the Necking-down Action of Steel Bars before Rupture.  
(Tetmajer, vol. 4.)

the length been increased by the load drawing out on the section where failure will finally occur. The first elongation, therefore, is that portion which is uniformly distributed over the specimen, and the second is that which is concentrated in the vicinity of the final failure. Both of these elongations are, however, measured and included in the total elongation, from which the percentage of elongation is determined. The total elongation is obtained after rupture has occurred, by placing the two ends together and measuring the distance between the primitive gauge-marks. In the case of specimens having shoulders at their ends the gauge-marks should be at least  $\frac{1}{2}$  in. inside of the shoulder, since the metal adjacent to the shoulder does not elongate fully, because of the strengthening effect of the enlarged cross-section at the ends.

It will at once be apparent from a study of these specimens that the

(b) elongation, or that which is locally developed in the vicinity of final rupture, is nearly the same in all these specimens; whereas the (a) elongation, or that which is uniformly distributed over the specimen, is always directly proportional to the length. The total elongation therefore, will not be proportional to the length. In other words, the percentage of total elongation will be greater for the short specimen than for the long ones. This shows the necessity of using standard lengths of these specimens when the percentage of elongation is to be found.

The percentage of elongation is the result which indicates the ductility of the material, this being one of the most important qualities of the metals used in structural designing.

**15. The reduction of area of cross-section** is found by determining the area of the broken cross-section, subtracting this from the original area of cross-section, and dividing the difference by the original area. This is not so important an indication or result as the others described above, but it is customary to determine it, and to add it to the record. For the ductile metals this reduction of area may be as much as from 50 to 60 per cent of the original cross-section.

**16. Failure in Tension.**—Illustration of the types of failure common to brittle and ductile metals may be seen in Fig. 6, Chapter III. In general, for ductile homogeneous materials the tensile elastic limit is reached when the shearing stress on any plane through the bar reaches the shearing elastic limit. Evidence of this is seen in the appearance of fine lines called Luders lines on the surface making angles of approximately  $45^\circ$  with the axis of the test-piece. The fracture of a ductile bar, like soft steel, shows a full cup and cone, the base angle of the latter also being about  $45^\circ$ . In the medium steels the cone is truncated, showing that the failure is partly shear and partly tensile. For very hard steels and other brittle materials the fracture is square across, showing that failure is due to tension.

Since the unit stress in shear on a  $45^\circ$  plane is  $\frac{P}{2A}$  it follows that we may expect a cone or truncated cone fracture whenever the ultimate shearing strength is less than half of the *true* tensile strength \* (i.e., tensile load

\* If we cut an oblique section through a bar under tension or compression and place upon the cut section equilibrating forces normal and tangential to it, the magnitude of the tangential component is  $P \cos \theta$  and the intensity of shear stress is

$$s_s = P \cos \theta \div \frac{A}{\sin \theta}, \quad \text{or} \quad S_s = \frac{P}{2A} \sin 2\theta;$$

where  $P$  = end load,  $A$  = area of cross-section, and  $\theta$  = inclination of cut to axis of bar.

$S_s = \frac{P}{2A}$ , a maximum value when  $\theta = 45^\circ$ .

divided by minimum area). When the true ultimate tensile strength is less than twice the shearing strength a square break will obtain.

Peculiarities in fracture will be further considered under tests of various materials.

### MATERIALS UNDER COMPRESSIVE STRESS

**17. Two Classes of Engineering Materials.**—Engineering materials may be divided into two general classes, according to their manner of failure in compression.

Plastic or viscous materials are those which will flow without showing any other indication of failure.

Brittle or comminuble materials are those which will crush to a powder, or crumble to pieces, or fail by shearing on definite angles under a compressive load.

In the former class are such materials as wrought iron, soft and medium steel, the alloys, lead, copper, zinc, and the like. Of the latter class are cast iron, hard or tempered steel, brick, stone, cement, etc. The laws of failure of these two classes are very different, and they will, therefore, have to be discussed separately.

**18. Crushing Strength of Plastic or Viscous Materials.**—There is no such thing as an "ultimate strength," in compression of a plastic body. There is, however, a definite yield point, the same as in tension. Beyond this limit the material simply spreads, and increases the area of its cross-section indefinitely under an increasing load, as shown in Fig. 11, Chapter III. This elastic limit in compression for wrought iron and steel is, fortunately, about the same in pounds per square inch as the elastic limit in tension. It is not customary, therefore, to test such materials in compression, but to assume that they have the same elastic limit in compression which they are found to have in tension.

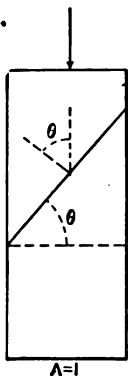


FIG. 7.

**19. The Law Governing the Strength in Compression of a Brittle or Comminuble Material.**—Experiments show that all such materials when subjected to a compressive load fail by shearing on certain definite angles. The resistance to movement along these angles is made up of two parts: first, the strength of the material to resist shearing; and second, the frictional resistance to motion along this plane. The sum of these two resistances must equal the shearing component of the load imposed when resolved along the shearing plane. To find what this angle should be, we may equate the two resistances here described with the shearing force, and find the angle of rupture, the determining condition being that this angle shall be that which offers the least total resistance to failure

under a crushing load. This angle may be found in the following manner:

Let  $S_s$  = shearing strength of the material per square inch;

$A$  = area of prism = 1 sq.in.;

$\theta$  = angle of rupture;

$S_c$  = crushing load per square inch.

The tendency to slide on the plane of rupture is  $S_c \sin \theta$ .

The resistance to sliding is  $S_s \sec \theta + f S_c \cos \theta$ , where  $f$  is the coefficient of friction =  $\tan \phi$ , where  $\phi$  = angle of repose. Hence, at failure,

$$S_c \sin \theta = S_s \sec \theta + f S_c \cos \theta. \quad (6)$$

It is evident that the angle of rupture will be such as to cause failure under the least load; hence if  $\theta$  be taken as the independent variable, we shall have at rupture

$$\frac{dS_c}{d\theta} = -S_s(\cos^2 \theta - \sin^2 \theta + 2f \sin \theta \cos \theta) = 0,$$

or

$$f = -\frac{\cos^2 \theta - \sin^2 \theta}{2 \sin \theta \cos \theta} = -\frac{\cos 2\theta}{\sin 2\theta} = -\cot 2\theta. \quad (7)$$

Whence, since  $f = \tan \phi$ , we have

$$\tan \phi = -\cot 2\theta = -\tan (90^\circ - 2\theta) = \tan (2\theta - 90^\circ),$$

or

$$\phi = 2\theta - 90^\circ \quad \text{and} \quad \theta = \frac{90^\circ + \phi}{2} = 45^\circ + \frac{\phi}{2}. \quad (8)$$

That is to say, the angle of rupture is  $45^\circ$  plus one half the angle of repose.

If the friction had been omitted, we should have had

$$S_c \sin \theta = S_s \sec \theta; \quad \text{whence} \quad \frac{dS_c}{d\theta} = -S_s(\cos^2 \theta - \sin^2 \theta) = 0;$$

whence

$$\cos^2 \theta = \sin^2 \theta, \quad \text{or} \quad \theta = 45^\circ. \quad (9)$$

It has been customary to neglect the friction, and to state that the planes of rupture make this angle of  $45^\circ$  with the horizontal; \* but the actual plane of rupture, when the specimen has sufficient height, is about  $55^\circ$  with the horizontal, or  $35^\circ$  from the direction of the applied load. (See Fig. 12, Chapter III, showing tests on sandstone made by Prof. Bauschinger.) Mr. Charles Bouton has shown † that the theoretical angle of

\* *Coulomb* is responsible for this theory, while *Navier* has given the true analysis. Most writers, including *Rankine*, have followed *Coulomb*, however.

† In a thesis for the degree M.S. at Washington University, 1981, entitled *Theory and Experiments on the Laws of Crushing Strength of Short Prisms*.

rupture is borne out in practice with many kinds of materials. (See Fig. 13, Chapter III, for photographic views of crushed specimens of cast-iron cylinders of various heights, showing angle of rupture.)

The following table gives the results of Mr. Bouton's determinations of the theoretical and the actual values of this angle:

Material.	Number of Experiments.	Observed Angle of Rupture. $\theta$	Observed Angle of Repose. $\phi$	Theoretical Angle of Rupture. $45^\circ + \frac{\phi}{2}$	Differences.
"F." cast iron.....	24	$54^\circ.8 \pm 0^\circ.2$	$20^\circ.6$	$55^\circ.3$	$-0^\circ.5$
"C. W." cast iron.....	24	$55^\circ.0 \pm 0^\circ.2$	$16^\circ.9$	$53^\circ.4$	$+1^\circ.6$
Limestone.....	4	62.2	33.4	61.7	$+0^\circ.5$
Asphalt paving mixture.	3	59.7	27.3	58.6	$+1^\circ.1$
Milwaukee brick.....	4	58.2	27.0	58.5	$-0^\circ.3$

The "F." cast iron was good foundry iron, having a tensile strength of 22,000 lb. per square inch and a modulus of elasticity of 14,500,000 lb. per square inch; the "C. W." iron was car-wheel iron, having a tensile strength of 20,000 lb. per square inch and a modulus of elasticity of 6,500,000 lb. per square inch, or less than one-half of the former.

**20. Relation of Crushing Strength to Shearing Strength.**—To show the relation of the crushing strength to the shearing strength, we have, from equation (6) in the previous article,

$$S_s = S_c(\sin \theta \cos \theta - f \cos^2 \theta);$$

also, from equation (7),

$$f = -\cot 2\theta = -\frac{\cos 2\theta}{\sin 2\theta} = -\frac{\cos^2 \theta - \sin^2 \theta}{2 \sin \theta \cos \theta}.$$

Substituting this value of  $f$ , we find

$$S_s = \frac{S_c \cos \theta}{2 \sin \theta} = \frac{1}{2} S_c \cot \theta, \quad \dots \dots \dots (10)$$

or

$$S_c = 2S_s \tan \theta, \quad \dots \dots \dots (11)$$

This relation was also shown by Mr. Bouton to be well borne out in tests.

**21. Column Action.**—If the length of a compression member is more than ten times its least lateral dimension the member is likely to bend and the intensity of stress on the concave side of it will be augmented by the bending stress which arises from the eccentricity of the load. Strictly the liability to lateral bending is dependent primarily upon the ratio of the length of the column to the least radius of gyration of the cross-section

—the slenderness ratio. In the short columns, where the slenderness ratio is less than 125, the effect of the lateral bending is of small moment; but in long columns it may be the controlling factor. Other factors which influence column action are the condition of the ends, the homogeneity of the member, stiffness, and the position of the load with respect to the center of resistance of the column.

There are four types of end conditions which are common: Round end—the end is free to rotate; fixed end—the axis of the column has a fixed position near the column end; square end—end of column and abutting surface are perpendicular to the axis; pin end—rotation of the column end is permitted in one plane only. Fig. 8 illustrates the behavior of a column under these different end conditions. These theoretical conditions are not realized in practice. In the round and pin-end columns friction

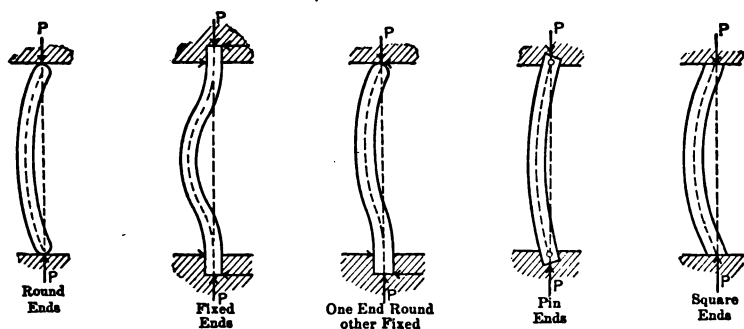


FIG. 8.—Effects of Various Types of End Conditions on the Bending of Columns.

at the bearings produces restraint; on the other hand no support is sufficiently rigid to produce a fixed end.

Owing to the uncertainty regarding conditions at the ends, regarding action between parts, and regarding the position of the load with respect to the axis of the column, theoretical analyses of column action are more or less rough and the main reliance of the designer is the information gotten from tests. The more important formulæ for columns under axial loading will now be very briefly considered.

*Euler's Formula.*—For straight and homogeneous long columns under axial loading a rational formula is that derived by Euler

$$\frac{P}{A} = \frac{mE}{\left(\frac{l}{r}\right)^2} \quad \cdot \cdot \cdot \cdot \cdot \cdot \cdot \quad (12)$$

where  $P$  = critical load which produces failure of the column by lateral bending.  $A$  = area of cross-section,  $m$  = a constant depending on end conditions (theoretically  $m = \pi^2$  for round ends,  $4\pi^2$  for fixed ends, and



$2.05\pi^2$  for one end fixed and the other round. With conditions of practice,  $m = 2\frac{1}{2}\pi^2$  for square ends and  $1\frac{1}{2}\pi^2$  for pin ends),  $E$  = modulus of elasticity and  $\frac{l}{r}$  = slenderness ratio. Difficulty in using this formula arises from the fact that it contains no term related to the strength of the material. Since it considers failure due to lateral flexure only, it cannot be used to design or investigate short columns. Whenever the value of  $\frac{P}{A}$  computed by this formula exceeds one-third of the ultimate compressive strength of the material the result should be rejected and another column formula used. (See Straight Line Formula and Fig. 9). Safe loads are derived from this formula by dividing  $P$  by a suitable factor of safety.

*Rankine's Formula.*—A semi-rational formula which holds in a very rough way for a wide range of slenderness ratios is

$$\frac{P}{A} = \frac{S}{1 + \phi \left(\frac{l}{r}\right)^2}, \quad \dots \dots \dots (13)$$

where  $P$  = ultimate load,  $A$  = area of cross-section,  $S$  = the ultimate compressive strength of a short prism,  $\phi$  = an imperial constant depending upon end conditions and kind of material, and  $\frac{l}{r}$  = slenderness ratio. Values of  $\phi$  recommended by Merriman are:

Material.	Both Ends Fixed.	Both Ends Round.	One End Round, One Fixed.
Timber.....	$\frac{1}{3000}$	$\frac{4}{3000}$	$\frac{1.95}{3000}$
Cast iron.....	$\frac{1}{5000}$	$\frac{4}{5000}$	$\frac{1.95}{5000}$
Wrought iron.....	$\frac{1}{36000}$	$\frac{4}{36000}$	$\frac{1.95}{36000}$
Steel.....	$\frac{1}{25000}$	$\frac{4}{25000}$	$\frac{1.95}{25000}$

Values of the safe load are obtained by dividing  $P$  by a proper safety factor. The formula cannot be used to investigate the unit stress on the concave side of the column when the load is within the elastic limit because  $\phi$  and  $S$  are usually determined from rupture tests.

*Straight Line Formula.*—T. H. Johnson worked out a straight line formula which, in conjunction with Euler's formula, gives results approxi-

rating the breaking values gotten by Tetmajer from experiments on medium steel struts. The formula is

$$\frac{P}{A} = S - C \frac{l}{r} \quad \dots \dots \dots (14)$$

Here  $C$  = the slope of the tangent to Euler's curve at the point where  $\frac{l}{r} = \left(\frac{3mE}{S}\right)^{\frac{1}{2}}$  and  $\frac{P}{A} = \frac{S}{3}$ , or  $C = \frac{2}{3}S\left(\frac{S}{3mE}\right)^{\frac{1}{2}}$ ;  $P$  = the ultimate load;  $A$  = area of cross-section;  $S$  = ultimate strength of short prism;  $\frac{l}{r}$  = slenderness ratio; and  $m = \pi^2$ ,  $1\pi^2$  and  $2\frac{1}{2}\pi^2$  for round, hinged and flat ends, respectively. Fig. 9

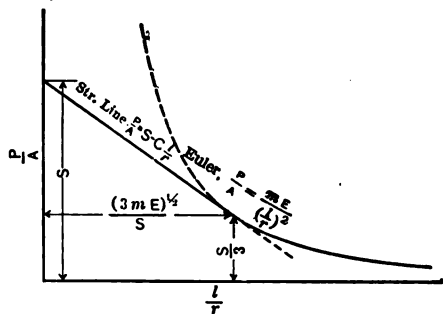


FIG. 9.

shows the straight line and the tangent Euler curve. Johnson's constants for his formula are given below.

Kind of Column.	$S$ , lb./in. <sup>2</sup>	$C$ , lb./in. <sup>2</sup>	Limit of $\frac{l}{r}$ .
Wrought iron:			
Flat ends.....	42,000	128	218
Hinged ends.....	42,000	157	178
Round ends.....	42,000	203	138
Structural steel:			
Flat ends.....	52,500	179	195
Hinged ends.....	52,500	220	159
Round ends.....	52,500	284	123
Cast iron:			
Flat ends.....	80,000	438	122
Hinged ends.....	80,000	537	99
Round ends.....	80,000	693	77
Oak:			
Flat ends.....	5,400	28	128

The straight line type of column formula has, on account of its simplicity, gained considerable favor among architects and engineers; and many such formulæ are found in specifications and in the building laws of our cities.

*The Parabolic Formula.*—From a thorough study of the results of a large number of tests by M. Considère on small steel bars, from tests by Tetmajer on a variety of steel and iron sections and from his own tests on timber, J. B. Johnson concluded that the strength of short columns

was limited by the yield point of the material.\* He proposed the following formula

$$\frac{P}{A} = S_y - f \left( \frac{l}{r} \right)^2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (15)$$

This is the equation of a parabola which is tangent to Euler's curve if  $f = \frac{S_y^2}{mE}$ . The ordinate of the point of tangency is  $\frac{S_y}{2}$  and the corresponding abscissa is  $\left( \frac{mE}{2S_y} \right)^{\frac{1}{2}}$ . In the above expressions  $P$  = ultimate load;  $A$  = area

of cross-section:  $S_y$  compressive yield-point;  $\frac{l}{r}$  = slenderness ratio;  $m = 1.6\pi^2$  and  $2.5\pi^2$  for columns with hinged and flat ends, respectively; and  $E$  = the modulus of elasticity in compression. Constants for Johnson's parabolic formula follow. For rectangular wooden columns the  $\left( \frac{l}{r} \right)^2$  term is replaced by its equivalent  $\left( \frac{l}{d} \right)^2$ , where  $d$  is the least lateral dimension, and  $f$  is also modified. Thus for white oak columns with flat ends the formula, in accordance with the tabulated values, is  $\frac{P}{A} = 3500 - 0.8 \left( \frac{l}{d} \right)^2$ .

In designing, the values of  $\frac{P}{A}$  should be divided by a suitable factor of safety. Formulæ of the parabolic type in combination with the proper Euler formulæ appear to be more accurate than any which thus far have been devised.

Kind of Column.	$S_y$ , lb./in. <sup>2</sup>	$f$ , lb./in. <sup>3</sup>	Limiting $\frac{l}{r}$ .	Value of $\frac{l}{d}$ .
Mild steel:				
Pin ends.....	42,000	0.97	150	
Flat ends.....	42,000	0.62	190	
Wrought iron:				
Pin ends.....	34,000	0.67	170	
Flat ends.....	34,000	0.43	210	
White pine:				
Flat ends.....	2,500	0.60	.....	60
Short-leaf yellow pine:				
Flat ends.....	3,300	0.70	.....	60
Long-leaf yellow pine:				
Flat ends.....	4,000	0.80	.....	60
White oak:				
Flat ends.....	3,500	0.80	.....	60

\* A conclusion which the results of recent tests on large columns at the Bureau of Standards Laboratory reaffirms. See *Engr. News*, Vol. 75, p. 190; also Vol. 76, pp. 49 and 81.

## MATERIALS UNDER SHEARING STRESS

**22. Two Manifestations of Shearing Stress.**—When all the opposing external forces which act on a body lie in one plane,\* but not in the same line, the resisting stresses are those of simple shear and cross-bending, without torsional stress.

When the opposing external forces do not lie in one plane the resisting stresses are those of torsional shear, with or without cross-bending and simple shear.

In any case these three kinds of stress are determined separately, as follows:

(a) *For Parallel External Forces in One Plane.*—The moment of resistance of the bending (direct) stresses at any transverse section is equal to the algebraic sum of the moments of the external forces on either side of that section taken about the neutral axis in that section.

The simple shearing stress on any section is equal to the algebraic sum of the transverse components of the external forces on either side of that section.

(b) *For Parallel External Forces Not in One Plane.*—First replace all the forces by equal parallel forces acting in the plane of the axis of the body, and by couples equal in value in each case to the force multiplied by its displacement. Then the moments of resistance and the simple shearing stresses will be the same as in the last case, and in addition there will be the moment of torsion.

The torsional moment at any transverse section is equal to the algebraic sum of the moments of the couples of the displaced forces, acting on either side of the transverse section in question.

(c) *For Non-parallel Forces Acting in Any Manner.*—Resolve all forces into horizontal and vertical components at their points of application, and then solve for bending moments, and torsions at any section in these two planes.

The bending moment at this section will then be the square root of the sum of the squares of the bending moments at right angles to each other.

The total shear will also be the square root of the sum of the squares of the primary shears at right angles to each other.

The total moment of torsion will be the algebraic sum of the two moments of torsion found from the two sets of forces.

**23. Shearing Stress Due to Torsion.**—In a solid or hollow member of circular cross section the twisting moment produces shearing deformations which, at any transverse section, vary from zero at the axis to a maximum on the surface. If the member is not deformed beyond its

\* When a force is distributed over an area it is here supposed to act at the center of gravity of these force-elements.

elastic limit the shearing deformations vary directly as the distance from the center of the cross-section, consequently the intensity of stress varies in a like manner, Fig. 10a.

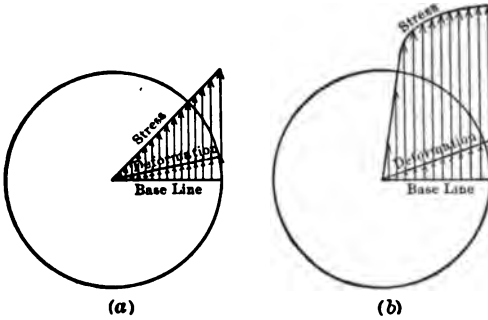


FIG. 10.—The Relation of Unit Shear Stress to Unit Deformation in a Solid Round Shaft (a), Stresses Within Elastic Limit (b), Overstrained.

If  $S_r$  is the intensity of stress on the outer fiber of a shaft having a radius  $r$ , the moment of stress on any element  $da$  at  $z$  distance from the axis is  $\frac{S_r z^2}{r} da$  and the total moment of all stresses is  $\frac{S_r J}{r}$ , where  $J$  is

the polar moment of inertia of the cross-section about the axis. This is the resisting torsional moment and must equal the external twisting moment  $M_t$ , or

$$M_t = \frac{S_r J}{r} \quad (16)$$

For a solid round shaft  $J = \frac{1}{2}\pi r^4$ , and for a hollow shaft with inner radius  $r_1$ ,  $J = \frac{1}{2}\pi(r^4 - r_1^4)$ .

Formula 16 does not hold exactly for sections other than circular.

For other sections approximate values of the factor  $\frac{J}{r}$  may be computed from the radius of the inscribed circle. Thus for a square shaft, with sides  $= d$ , the exact analysis gives  $M_t = 0.2083 S d^3$  which is about 5.9 per cent greater than the value for a solid round shaft of diameter  $d$ . For a solid elliptical shaft of major axis  $a$  and minor axis  $b$  the exact value for  $M_t = \frac{\pi}{16} S a b^2$ . If  $a = 2b$  then the exact value of  $M_t$  is double that for a round shaft of diameter  $b$ . Similarly for a rectangular shaft  $M_t = \frac{a^2 b^2 S}{3a + 1.8b}$ , where  $a$  is the length of the long side and  $b$  the length of the short side. The greatest intensity of stress in an elliptical or rectangular shaft occurs at the ends of the minor axis.

If the shaft is deformed beyond its elastic limit the shearing deformation increases approximately as the distance of the fiber from the axis but the intensity of stress does not vary directly with the unit shearing deformation, Fig. 10b. Values of the ultimate shearing stress in torsion cannot, therefore, be computed from  $S_s = \frac{M_t r}{J}$ . Upton \* has made a

\* See *Materials of Construction* by Upton, p. 52.

mathematical analysis by which the true intensity of shearing stress in a solid round shaft may be gotten as follows: In Fig. 11 the full-line curve represents the relation of unit stress  $S_s$  on the outside fiber to the unit deformation of the same fiber. In plotting this curve  $S_s$  is computed by formula 16 and  $\epsilon_s$  must be determined from experiment. The true stresses are represented by ordinates to the dotted line. To find the true stress  $DB'$  corresponding to a unit shearing deformation on the surface  $=AD$ , draw  $BD$  perpendicular to  $AD$ , prolong the tangent at  $B$  until it intersects the  $S_s$  axis at  $C$ ; then

$$DB' = DB - \frac{AC}{4}.$$

For ductile materials the shearing stress-deformation curve is approximately parallel to the deformation axis at the maximum  $S_s$ , therefore the true ultimate shearing stress is

$$S'_s = \frac{3}{4} \frac{M_t r}{J} = .477 \frac{M_t}{r^3}. \quad (17)$$

For hollow shafts of ductile material in which the thickness is less than a fifth of the outer radius the intensity of shearing stress is approximately uniform throughout the cross-section when the shaft is stressed to the ultimate, therefore the maximum unit shearing stress is approximately

$$S_s = \frac{2M_t}{\pi(r+r_1)^2(r-r_1)}. \quad (18)$$

**24. Shearing Deformations.**—As shown in Art. 7, a shearing action of external forces results in angular deformation of the body. In the

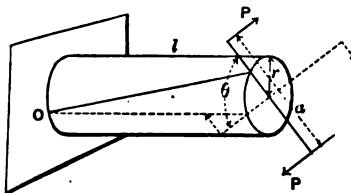


FIG. 12.

case of simple shear, or where the forces lie in one plane, the angular deformation from shear is very small, the bending being mostly due to the longitudinal deformations resulting in the direct tensile and compressive resisting stresses on the two sides of the neutral plane respectively.\* When the forces do not lie in one plane, or when there is a moment of

\* Shear in beams will be discussed in Art. 28.

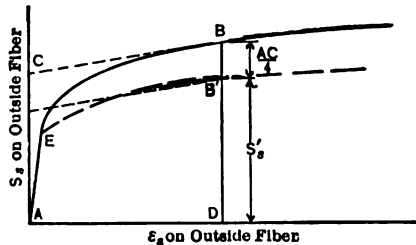


FIG. 11.—Method of Finding True Unit Shear Stress in an Overstrained Solid Round Shaft (Upton).

Then  $S_s = \frac{M_s r}{J} = \frac{2Pa}{\pi r^3}$ . From Fig. 12 it appears that the unit shearing deformation of an outside fiber  $\epsilon_s = \frac{r\theta}{l}$ . Also from Art. 7  $\epsilon_s = \frac{S_s}{E_s}$ , therefore by combining these three equations there results

$$\theta = \frac{M_s l}{E_s J} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (19)$$

Here  $\theta$  is expressed in radians. To obtain the angle of twist in degrees multiply  $\theta$  by 57.3. Formula 19 holds for hollow or solid round shafts provided the stress on the extreme fiber does not exceed the elastic limit.

The angle of twist in radians for a square shaft of side  $d$  is  $\theta = 7.11 \frac{M_s l}{E_s d^4}$ , or 43 per cent more than  $\theta$  for a solid round shaft. For an elliptical section  $\theta = \frac{4\pi^2 M_s J l}{A^4 E_s}$ , where  $\theta$  is in radians.  $A$  is the area of cross-section, and  $J$  is the polar moment of inertia. This is also an approximate formula for the twist of rectangular shafts.

The fracture of ductile materials under torsion is generally square across. Wrought iron and some of the brasses show a rope-like twisting of the fibers before a square break ensues. Brittle materials like cast iron, stone, brick and concrete exhibit a helicoid fracture. These substances being weak in tension really fail through the secondary tensile stresses which are produced by two pairs of shearing stresses acting in planes tangent to the surface of the shaft. One of these pairs of forces acts perpendicular to the surface elements; the other pair acts parallel to the elements. Consequently on a plane at  $45^\circ$  with the axis of the shaft, there is produced a unit tensile stress  $S_t$  which is equal to the intensity of the shear stress  $S_s$  (Art. 7).

## MATERIALS UNDER CROSS BENDING STRESS

**25. Fundamental Principles.**—When a member is bent by forces applied transversely with respect to its axis, cross-bending stresses are produced. The simplest system of forces which will cause such stresses is a coplanar parallel system acting in a plane containing the longitudinal axis of the member. Such a member is called a beam. Frequently beams are horizontal, the loads are produced by gravity pulls on suspended masses and the supporting forces are upward. Under the action of these forces the beam is bent and observation shows that the fibers on its convex surface are elongated, those on its concave surface are shortened, and that there must be a plane of fibers between the convex and concave surfaces which suffers no deformation. This plane is called the neutral

surface. Hence tension exists in the fibers between the neutral and the convex surfaces and compression exists in the fibers between the neutral and the concave surfaces of the beam.

It will be assumed that the beam is symmetrical, initially straight, homogeneous, of material having equal stiffness in tension and compression, that it is not stressed beyond its elastic limit by the loads, that the bending is slight, and that the plane of the external forces coincides with a plane of symmetry. If the beam is severed transversely it will be necessary, in order that equilibrium may obtain, to place a system of forces on either cut surface similar to that shown in the lower part of Fig. 13. Then from the principles of Statics we know that:

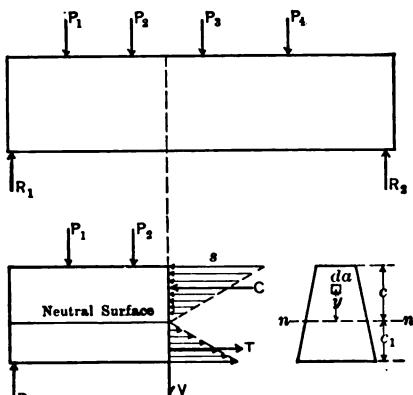


FIG. 13.

1. The vertical shear  $V$  equals the sum of the vertical forces on the left of the cut;
2. The total compression  $C$  equals the total tension  $T$ ;
3. The sum of the moments of the tensile and compressive stresses on the portion shown equals the algebraic sum of the moment of the external forces to the left of the section.

From experimental evidence it is known that the unit deformations vary directly as the distance of the fiber from the neutral surface, consequently the intensity of stress varies in like manner.

From these considerations it is easily shown that the total horizontal stress on the cut is  $0 = \frac{S}{c} \sum_{c_1}^c y da$ , where  $S$  is the unit stress on the extreme fiber at  $c$  distance from the neutral axis,  $y$  is the distance from the neutral axis to any elementary area (Fig. 13) and the summation is taken for the entire cross-section. Since  $\frac{S}{c}$  is finite it follows that  $\sum_{c_1}^c y da = 0$ , or the neutral axis must pass through the center of gravity of the cross-section.

**26. Resisting Moment Equals Bending Moment.**—Taking moments of the stresses about the trace of the neutral surface ( $n-n$ ) and using








notation, we have  $M_R = \frac{S}{c} \sum_{c_1}^c y^2 da = \frac{S}{c} I$ . Since the resisting moment ( $M_R$ ) equals the bending moment ( $M$ ) there results

$$M = \frac{SI}{c}. \quad \dots \dots \dots (20)$$



For selecting a beam to carrying a given loading  $S$  is known and  $M$  can be computed by taking the algebraic sum of the moments of all forces on either side of the dangerous section, therefore  $\frac{I}{c}$  may be ascertained and the section designed. To determine the safe load for a given beam;  $S$ ,  $I$  and  $c$  are known or may be computed and  $M$  is expressed in terms of the unknown load which can then be found. The third type of problem consists in finding the unit stress at a certain section of a given beam. In this problem  $M$  is computed for the given section;  $I$  and  $c$  are known and  $S$  can be determined.

Values of the moment of inertia and resisting moment for several common beam sections appear in the following table.

Form of Cross-section.	Distance of Center of Gravity, or Neutral Axis, from the Most Distant Fiber. $= c$	Moments of Inertia about the Center of Gravity of the Section. $= I$	Moment of Resistance in Terms of the Stress in the Most Distant Fiber. $= M_r = \frac{SI}{c}$
	$\frac{d}{2}$	$\frac{bd^3}{12}$	$\frac{1}{6}Sbd^2$
	$\frac{d}{2}$	$\frac{\pi d^4}{64}$	$\frac{\pi}{32}Sd^3$
	$\frac{2}{3}h$	$\frac{bh^3}{36}$	$\frac{1}{24}Sbh^2$
	$\frac{d}{2}\sqrt{2}$	$\frac{d^4}{12}$	$\frac{1}{6\sqrt{2}}Sd^3$
	$\frac{h}{2}$	$\frac{bh^3 - (b - t')(h - 2t')^3}{12}$	$\frac{bh^3 - 6b - t')(h - 2t')^3}{6h}S$
	$\frac{\frac{1}{2}t'h^3 + t'(b - t')(h + \frac{1}{2}t')}{t'h + t(b - t')}$	$\frac{bh^3 - (b - t')(h - t')^3}{3} - Ac^2$	$\frac{SI}{c}$
	$\frac{b + 2b'}{b + b'} \cdot \frac{h}{3}$	$h^3 \left[ \frac{3b + b'}{12} - \frac{(b + 2b')^2}{18(b + b')} \right]$	$\frac{Sh^2}{6} \left[ \frac{3(3b + b')(b + b')}{2(b + 2b')} - (b + 2b') \right]$

When the plane of the loads does not contain an axis of symmetry of the section, then the neutral axis is not in general perpendicular to the plane of the loads and the above formulæ are inexact. In such cases the principal axes of the section (see Moment of Inertia in Applied Mechanics) may be determined; the bending moment is resolved into the planes of these axes; and the unit stress on a given fiber is found by adding algebraically the stress due to each moment considered separately. In finding the stress due to a component moment, the neutral plane for that moment is the plane of the principal axis perpendicular to that moment. Thus if  $x$  and  $y$  are the principal axes of the section and the  $z$ -axis coincides with the longitudinal axis of the beam,

$$S_1 = \frac{M_y y_1}{I_z} + \frac{M_x x_1}{I_y}$$

Where  $S_1$  is the unit stress on fiber whose coordinates are  $x_1$  and  $y_1$  with respect to the principal axes,  $M_x$  is the component of the bending moment in the  $xz$ -plane,  $M_y$  is the component in the  $yz$ -plane,  $I_z$  is the moment of inertia about the  $z$ -axis and  $I_y$  about the  $y$ -axis.

**27. Stresses in Overstressed Beams.**—The flexure formula 20 does not hold for beams of materials where  $E_c$  does not equal  $E_t$ , nor for beams stressed beyond the elastic limit. In the latter case the intensity of stress

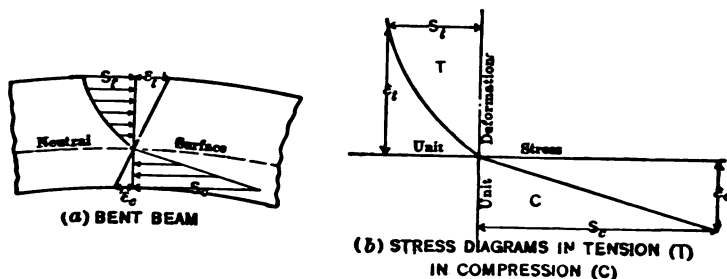


FIG. 14.

does not vary as the distance of the fiber from the neutral axis but follows the law of the variation of the stress ordinates to a stress-diagram in which the extreme ordinate represents the stress on the extreme fiber of the beam. If the ultimate strengths in tension and compression are unequal, the neutral axis will shift toward the stronger side of the beam as the overstressing proceeds. Thus the stress variation in a beam of cast iron, which is much stronger in compression than in tension, is illustrated in Fig. 14. The unit deformation ( $\epsilon_t$ ) of the outside fiber in tension (Fig. 14a) corresponds to a unit stress  $S_t$  in the stress diagram of Fig. 14b. Similarly  $S_c$  the bottom fiber and the unit stresses for other fibers may be found. If the load is increased until  $S_t$  equals the ultimate tensile strength of the iron, failure begins. Concrete, brick, and stone beams fail similarly; a wooden beam, being weaker in compression, will fail first in compression.

If formula 20 be applied to find stresses beyond the elastic limit, it is evident from the foregoing that the results will be fictitious. In general the value of  $S_m$ , found when  $M_m$  is the maximum moment, will lie between

the ultimate tensile and compressive strengths for the material. It is called the modulus of rupture. If the shape and size of specimen is maintained approximately constant, the modulus of rupture furnishes a good index of the strength of different grades of material. For the effect of

variations in form on the modulus of rupture of cast iron beams see Art. 11.

Upton has also worked out the true value of the unit stress on the extreme fiber of an overstressed beam of rectangular cross-section.\* A curve is plotted as in Fig. 15 between the computed stresses on the extreme fiber at the given section and the unit deformations of the same fiber.

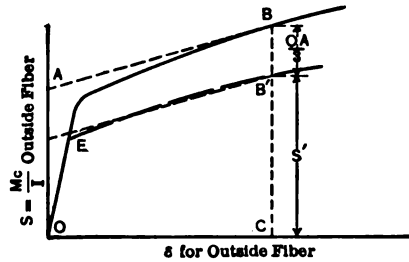


FIG. 15.—Method of Finding True Unit Stress in an Overstrained Beam (Upton).

The true unit stress  $S'$  corresponding to a given unit deformation  $OC$  is  $CB' = CB - \frac{OA}{3}$ ,  $AB$  being tangent to the curve  $OEB$  at  $B$ . Ordinates to the dotted curve  $EB'$  represents true unit stresses. Use of the above method necessitates measurement of the deflection and calculation of the values of  $\epsilon$ , or direct measurement of the latter which is cumbersome. For ductile materials the stress-deformation curve is approximately horizontal when the maximum moment ( $M_m$ ) is imposed, therefore the true stress on the extreme fiber is two-thirds of the computed stress or

$$S'_m = \frac{2}{3} \frac{M_m c}{I} = \frac{4M_m}{3bd^2} \quad (22)$$

### 28. Variation in the Intensity of Shearing Stress within a Beam.—

If a rectangular prism be cut from a beam and the forces necessary to

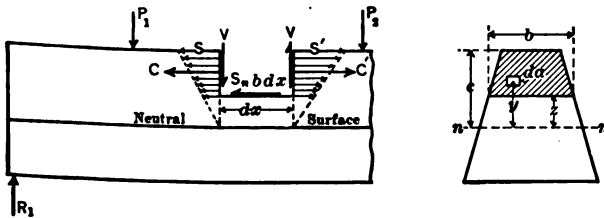


FIG. 16.

equilibrium be placed on the cut surfaces as shown in Fig. 16, the occurrence of a horizontal shearing stress acting along the surface which parallels the neutral plane is noted. We shall now determine the average † inten-

\* See *Materials of Construction*, p. 78.

† The intensity of shearing stress on a horizontal plane is not always uniform unless the cross-section is rectangular.

of shearing stress  $S_h$ . Let  $M$  be the bending moment at the left end of the notch,  $M'$  the bending moment at the right end,  $S$  and  $S'$  the stresses on the extreme fibers at the left and right end of the notch, respectively,  $C$  the total stress on the right end,  $C'$  the total stress on the left end of notch,  $b$  the breadth of the bottom of the notch, and  $I$  the moment of inertia of the cross-section about the neutral axis  $n-n$ . Then

$$C = \frac{S}{c} \sum y da \quad \text{and} \quad C' = \frac{S'}{c} \sum y da;$$

or 
$$C' - C = S_h b dx = \left[ \frac{S'}{c} - \frac{S}{c} \right] \sum y da;$$

$$\frac{S'}{c} - \frac{S}{c} = \frac{M'}{I} - \frac{M}{I} = \frac{dM}{I};$$

therefore 
$$S_h b dx = \frac{dM}{I} \sum y da;$$

$$S_h = \frac{dM}{dx I b} \sum y da. \quad \text{Since } \frac{dM}{dx} = V, \text{ we have}$$

$$S_h = \frac{V}{I b} \sum y da. \quad \dots \dots \dots (23)$$

In the above expression  $\sum y da$  is the statical moment of the shaded area in Fig. 16 about the neutral axis  $n-n$ . Since it can easily be shown that the intensity of the vertical and horizontal shearing stresses at any point in a beam must be equal to produce equilibrium (see Fig. 1), it follows that values of the vertical unit shearing stress  $S_v$  may also be gotten by formula (23).

For a rectangular beam the intensity of the horizontal shear at any point varies, in accordance with the ordinates to a parabola, from zero at the outside fibers to a maximum at the neutral axis. For such beams the maximum value of the horizontal or vertical unit shearing stress is

$$S_h = S_v = \frac{3V}{2bd} \quad \dots \dots \dots (24)$$

where  $b$  = the breadth and  $d$  the depth of the cross-section. That is, the maximum intensity is  $\frac{3}{2}$  of the mean intensity of shearing stress. Fig. 17 shows how the total shear and the intensity of the horizontal and vertical

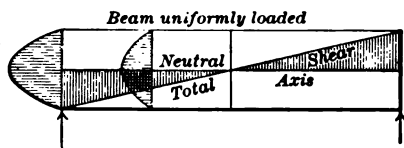


FIG. 17.

shearing stresses vary in a rectangular beam under uniform load. In a solid circular section the maximum intensity of shearing stress is  $\frac{3}{2}$  of the mean intensity.

For an I-beam or plate girder the maximum intensity of shear stress is practically equal to the total shear at the section divided by the area of the web.

*Shear in Wooden Beams.*—It becomes necessary to design wooden beams for horizontal shear when the safe load in shear is less than the safe load in bending. This condition occurs only in short deep beams. Thus for a uniformly loaded rectangular beam of simple span, the safe load in bending is  $W_b = \frac{4Sbd^2}{3l}$  and the safe load in shear is  $W_s = \frac{1}{2}S_hbd$ . If

$W_s < W_b$ , then must  $\frac{S_h}{d} < \frac{S}{l}$ . When the load is concentrated at the center, if  $W_s < W_b$ , then must  $\frac{S_h}{d} < \frac{S}{2l}$ . Here  $S$  and  $S_h$  are the allowable unit stresses in bending and horizontal shear, respectively, and  $b$ ,  $d$  and  $l$  are the breadth, depth and length of the beam in the same linear units as  $S$  is expressed.

**29. Deflection of Beams Due to Bending Moment.**—Let Fig. 18

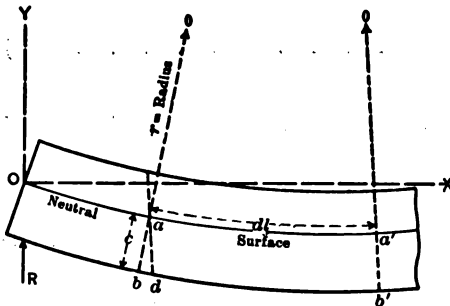


FIG. 18.

represent a portion of a bent beam. We shall now derive an expression for the radius of curvature and the differential equation by which the deflection of the neutral surface with respect to a set of axes may be determined. The elongation of the lower outer fiber of length  $dl$ , found by drawing  $ad$  parallel to  $a'b'$ , is  $bd$ . The unit elongation is  $\frac{bd}{dl} = \epsilon$  and the

unit stress in the fiber is  $S = \epsilon E$ . From the similarity of triangles  $aaa'$  and  $abd$  we have  $\frac{r}{dl} = \frac{c}{db}$ . Therefore  $r = \frac{c}{\epsilon} = \frac{cE}{S}$ . Since  $S = \frac{Mc}{I}$  we have finally

$$r = \frac{EI}{M} \quad (25)$$

From calculus  $r = \frac{[1 + (\frac{dy}{dx})^2]^{3/2}}{\frac{d^2y}{dx^2}}$ . Since in beams the curvature is slight

$(\frac{dy}{dx})^2$  is negligible compared to  $l$ , therefore approximately  $\frac{d^2y}{dx^2} = \frac{1}{r}$  Substituting in (equation 25) we have

$$EI \frac{d^2y}{dx^2} = M \quad (26)$$

In this equation one must know the relation of  $E$ ,  $I$  and  $M$  to the  $y$  and  $x$ . Commonly  $E$  and  $I$  are constants and  $M$  can be expressed in terms of the variable  $x$ . The first integration gives the slope of the beam at any point, and the second integration gives the deflection of the beam at any point. The first integration gives the slope of the beam at any point, and the second integration gives the deflection of the beam at any point.

To illustrate the use of the above equation, we shall determine the deflection of a beam with an end load, Fig. 19. The bending moment at any section  $x$  distance from the support is  $-P(l-x)$ ; hence

$$(a) \quad EI \frac{d^2 y}{dx^2} = -P(l-x);$$

$$(b) \quad EI \frac{dy}{dx} = -Plx + \frac{Px^2}{2} + c \quad [c=0, \text{ since } \frac{dy}{dx}=0 \text{ when } x=0];$$

$$(c) \quad EI y = -\frac{Plx^2}{2} + \frac{Px^3}{6} + c_1 \quad [c_1=0, \text{ since } x=0 \text{ when } y=0].$$

The deflection is a maximum when  $x=l$ .

$$\therefore y_{\max} = f = -\frac{Pl^3}{3EI} \quad (27)$$

For this loading is a maximum at the support and equals  $-Pl$ .

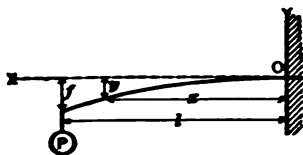


FIG. 19.

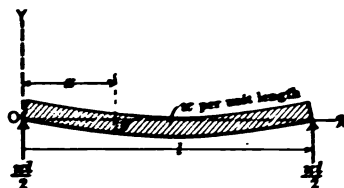


FIG. 20.

As a second illustration consider a simple beam with a uniform load, Fig. 20. The moment equation for any section  $x$ -distance from the left

is  $\frac{wx}{2} - \frac{wx^2}{2}$ ; hence

$$(d) \quad EI \frac{d^2 y}{dx^2} = \frac{wx}{2} - \frac{wx^2}{2};$$

$$(e) \quad EI \frac{dy}{dx} = \frac{wx^2}{4} - \frac{wx^3}{6} + c \quad \left[ \frac{dy}{dx}=0 \text{ when } x=\frac{l}{2} \therefore c = -\frac{wl^3}{24} \right];$$

$$(f) \quad EI y = \frac{wx^3}{12} - \frac{wx^4}{24} - \frac{wl^3 x}{24} + c_1 \quad [x=0 \text{ when } y=0 \therefore c_1=0].$$

The maximum deflection is found by substituting  $x = \frac{l}{2}$  in equation  $f$ ; therefore

$$y_{\max} = f = \frac{-5wl^4}{384EI} \quad \dots \quad (28)$$

The moment for this loading is a maximum at the center and equals  $\frac{wl^2}{8}$ .

The use of equation (26) becomes more tedious when the bending moment equation is a discontinuous function of  $x$ . In such cases it is generally necessary to determine the equation of the elastic curve for each portion of the beam. Elimination of the constants of integration is facilitated by remembering that the two slope equations for adjacent portions of the beam have a common value at the point of discontinuity; likewise the ordinate equations have equal values at this point. For the solution of the more complicated cases the student is referred to treatises on Applied Mechanics.

Values of the maximum moments and deflections for other beams under common types of loading appear in Table 1.

**30. Deflection of Beams Due to Shear.**—Besides the deflection due to bending moment there is a very slight deflection due to shear. This is of importance only in short deep beams where  $l < 10d$ . The shearing deflection may be closely approximated by equating the energy stored by the fibers in the beam to the work done by the load in moving through a distance equal to the shearing deflection.\* For a rectangular cantilever

with an end load ( $P$ ) the shearing deflection is  $f_s = \frac{6}{5} \frac{Pl}{E_s bd}$ ; for a rectangular simple beam with a load ( $P$ ) at the middle it is  $f_s = \frac{3}{10} \frac{Pl}{E_s bd}$ . For uniform

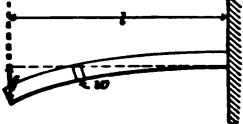
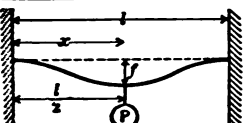
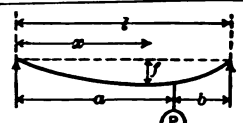
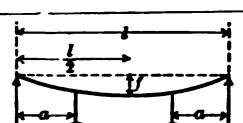
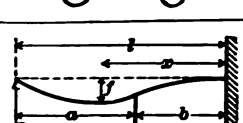
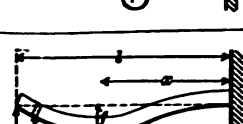
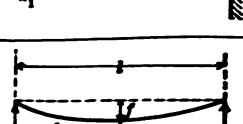
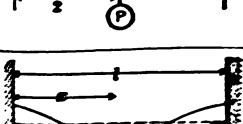
loads equal to  $P$  in each of the above cases the deflection is only  $\frac{1}{3}$  as much. From the above it appears that the deflection of a rectangular beam due to shear is to the deflection due to bending as  $\frac{Kd^2}{l^2}$  assuming

$\frac{E_s}{E} = \frac{2}{5}$  then  $K = \frac{3}{4}$  for a cantilever with an end load, 3 for a simple beam with a center load,  $\frac{3}{2}$  for a cantilever with a uniform load and 1.6 for a simple beam with a uniform load.

**31. Curved Beams.**—When the axis of a homogeneous beam is curved, bending of the beam produces stresses which cannot be accurately calculated by the formulæ of Art. 26. If the axis of the beam lies in a plane curve which cuts the principal axis of each cross-section and the forces act in that plane, it is sufficiently exact, however, to assume that the

\* See Morley's *Strength of Materials*, p. 226.

TABLE 1.—MAXIMUM MOMENTS AND DEFLECTIONS FOR BEAMS

Method of Loading.	Max. Moment, $M$ .	Max. Deflection, $f$ .
	$M = -\frac{wl^2}{2}$	$f = \frac{wl^4}{8EI}$
	$M = \frac{Pl}{4}$	$f = \frac{Pl^3}{48EI}$
	$M = \frac{Pab}{l}$	$f = \frac{Pb}{27EI} \sqrt{3[a(2b+a)]^3}$ for $x = \frac{1}{3} \sqrt{3[a(2b+a)]}$
	$M = Pa$	$f = \frac{Pa}{6EI} [l^3 - a^3]$
	$R_1 = \frac{P}{2l^2} [3lb^2 - b^3]$ $M = R_1a$ , pos. $M = R_1l - Pb$ , neg.	$f = \frac{Pb^2}{6EI} (l-b) \sqrt{\frac{l-b}{3l-b}}$ for $x = l \left( 1 - \sqrt{\frac{l-b}{3l-b}} \right)$ if $a > 0.414l$
	$R_1 = \frac{1}{2}wl$ $M = -\frac{wl^2}{8}$ for $x = 0$ $M = +\frac{9wl^2}{128}$ for $x = \frac{3l}{8}$	$f = 0.0064 \frac{wl^4}{EI}$ for $x = 0.578l$
	$M = -\frac{Pl}{8}$ for $x = 0$ $M = +\frac{Pl}{8}$ for $x = \frac{l}{2}$	$f = \frac{Pl^3}{192EI}$
	$M = -\frac{wl^2}{12}$ for $x = 0$ $M = -\frac{wl^2}{24}$ for $x = \frac{l}{2}$	$f = \frac{wl^4}{24EI}$



total deformations vary as the distances of the fibers from the neutr surface; but since the fibers are of unequal length the unit deformation

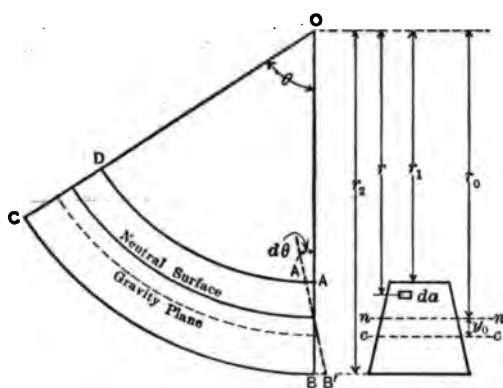


FIG. 21.

hence unit stresses, will not so vary. As a result of this condition the neutr surface does not, in general, include the gravity axis of the beam.

Let Fig. 21 represent a small portion of a beam.\* Consider the end CD fixed and suppose A' to be the position assumed by AB after bending. Then the unit deformation any fiber with cross-section

$da$  is  $\epsilon = \frac{rd\theta - r_0d\theta}{r\theta}$  and the unit stress  $\epsilon E = \frac{(r-r_0)Ed\theta}{r\theta}$ . The position of the neutral axis is obtained by equating the total stress on the cross-section to zero; or  $0 = \int_{r_1}^r \frac{(r-r_0)Ed\theta}{r\theta} da$ ; since  $\frac{Ed\theta}{\theta}$  does not equal zero but a constant for any given section we have

$$r_0 = \frac{A}{\int_{r_1}^r \frac{da}{r}} \quad (28)$$

The resisting moment is

$$M_R = \frac{Ed\theta}{\theta} \int_{r_1}^r \frac{(r-r_0)^2}{r} da \quad (29)$$

and the unit stress on the inner fiber due to a bending moment  $M$  is

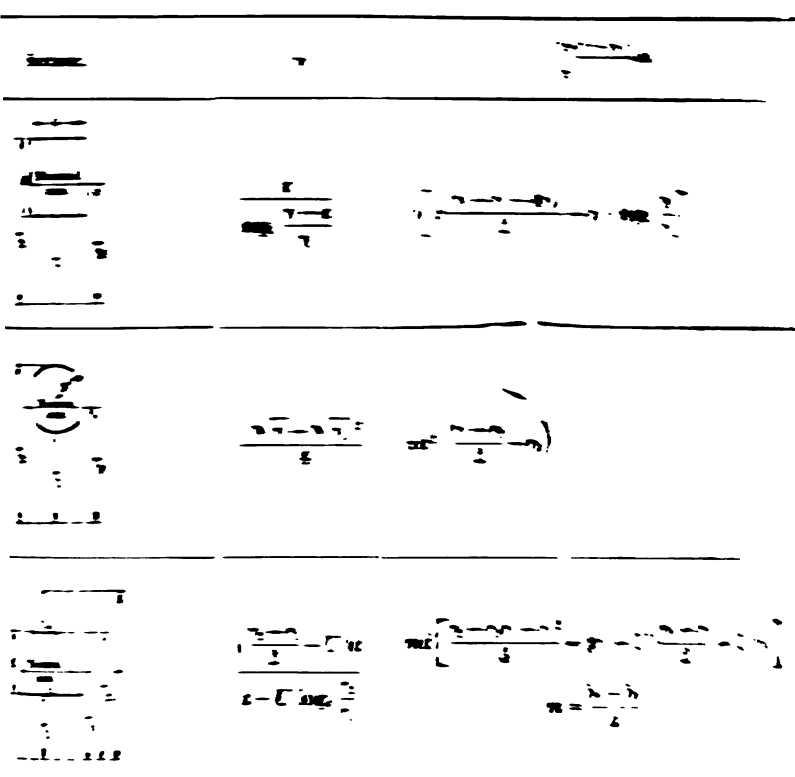
$$S_1 = \frac{(r_1 - r_0)M}{r_1} \frac{1}{\int_{r_1}^r \frac{(r-r_0)^2}{r} da} \quad (30)$$

On the outer fiber it is

$$S_2 = \frac{r_2 - r_1 M}{r_2} \frac{1}{\int_{r_1}^r \frac{(r-r_0)^2}{r} da} \quad (31)$$

In solving equations (29) to (32) the form of the section must be known in order that the relation of  $da$  to  $r$  may be properly inserted in evaluation.

\* The proof outlined is more completely given in Boyd's *Strength of Materials* Chapter XVIII.



accompanying table shows the ratios of unit stresses computed by eq. (31) and gotten from  $S = \frac{Mc}{I}$ , when the curvature is sharp.\*

\* Taken from Boyd's *Strength of Materials*, pp. 327 and 333.

# OF UNIT STRESS BY CURVED BEAM FORMULA TO UNIT STRESS BY STRAIGHT-BEAM FORMULA FOR EXTREME FIBERS

RECTANGLE		CIRCLE	
Concave.	Convex.	Concave	Convex
1.15	.87	1.17	.86
1.29	.81	1.33	.79
1.52	.73	1.62	.70

In many practical problems a curved beam is bent by forces having components which are perpendicular to the cross-section of the member. In such cases the values of  $r_0$  do not indicate the position of the true neutral surface, but the position which it would occupy if bending only obtained. The bending moment in such cases is computed with respect to an axis through the center of gravity of the section considered and includes the moments of all forces on one side of the given section. The unit stress on any fiber is equal to the bending stress computed as indicated above plus or minus  $\frac{P}{A}$ .

**Example.** A 4-by 2-in. rectangular bar is bent in an elliptical arc and loaded at either end with axial pulls of 3000 lb. The center of gravity of the remotest section is 5 in. from the line of action of the pulls. The inner radius ( $r_1$ ) of that section is 8 in. and the outer radius ( $r_2$ ) is 12 in. Find the unit stresses  $S_1'$  and  $S_2'$  on the inner and outer faces, respectively. Here  $A = 8$  in.<sup>2</sup>,

$$r_0 = \frac{d}{\log_e \frac{r_1 + d}{r_1}} = \frac{4}{0.1761 \times 2.3026} = \frac{4}{.4057} = 9.865 \text{ in.},$$

$$b \left[ d \frac{r_2 + r_1 - 4r_0}{2} + r_0^2 \log_e \frac{r_2}{r_1} \right] = -1.08 \text{ in.}^3$$

$$\text{From eq. (31)} \quad S_1 = \frac{8 - 9.865}{8} \frac{(-5 \times 3000)}{1.08} = 3240 \text{ lb./in.}^2$$

$$S_1' = \frac{P}{A} + S_1 = \frac{3000}{8} + 3240 = 3615 \text{ lb./in.}^2 \text{ tension.}$$

$$\text{From eq. (32)} \quad S_2 = \frac{12 - 9.865}{12} \frac{(-5 \times 3000)}{1.08} = -2470 \text{ lb./in.}^2$$

$$S_2' = \frac{P}{A} + S_2 = +375 - 2470 = 2095 \text{ lb./in.}^2 \text{ compression.}$$

**32. Approximate Determination of the Strength of Flat Plates under Normal Forces.\***—(a) *Flat Circular Plate Supported at the Circumference and Uniformly Loaded.*—Assume a diametral strip 1 in. in width to be loaded over its full width at the ends, but the loaded surface to reduce to a zero width at the center, this load to be  $w$  lb. per square inch. The total load on the strip will then be  $wr$ , and each end support will be  $\frac{wr}{2}$ . The bending moment at the center will be

$$M_0 = \frac{wr}{2} \cdot r - \frac{wr}{2} \cdot \frac{2}{3}r = \frac{wr^2}{6}. \quad \dots \dots (33)$$

\* These proximate solutions are offered as illustrative of simple approximate methods which may often be applied to very complicated problems of this class.

for a solid rectangular section we have

$$M_0 = \frac{1}{6} S b t^2, \text{ or, for } b, = 1;$$

$$M_0 = \frac{w r^2}{6} = \frac{S t^2}{6}, \text{ or } S = \frac{w r^2}{t^2}; \dots (34)$$

or

$$t = r \sqrt{\frac{w}{S}}, \dots (35)$$

- $t$  = thickness of plate in inches;  
 $r$  = radius of plate in inches;  
 $s$  = stress in extreme fiber in pounds per square inch;  
 $w$  = pressure on plate in pounds per square inch.

From a very elaborate analysis, Prof. Grashof finds for this case

$$t = r \sqrt{\frac{5}{6} \frac{w}{S}} = 0.91 r \sqrt{\frac{w}{S}}.$$

(b) *Square Flat Plate Supported at the Periphery and Uniformly Loaded.*

—Since the corners are more distant from the center and therefore carry a less proportion of the load, we may assume that the opposite sides act independently, so far as the bending moment at the center is concerned. On this assumption the plate may be regarded as supported at two sides only and loaded with one-half the actual load, whence we have

$$M_0 = \frac{1}{16} w b l^2 = \frac{1}{6} S b t^2, \dots (36)$$

or

$$t = l \sqrt{\frac{3}{8} \frac{w}{S}} = 0.61 l \sqrt{\frac{w}{S}}, \dots (37)$$

where  $l$  = length of one side of the square plate.

(c) *Same Cases when the Plates are Fixed in Position at Their Peripheries.*—Since the maximum bending moment on a beam fixed at the ends and uniformly loaded is only  $\frac{2}{3}$  that of a beam supported at the ends and similarly loaded, we may assume the same relations would hold here, thus giving for a circular plate, rigidly held,

$$S = \frac{2}{3} \frac{w r^2}{t^2}, \text{ or } t = r \sqrt{\frac{2w}{3S}}. \dots (38)$$

For a square plate, rigidly held,

$$S = \frac{1}{4} \frac{w l^2}{t^2}, \text{ or } t = \frac{l}{2} \sqrt{\frac{w}{S}}. \dots (39)$$

(d) *For Elliptical and Rectangular Plates.*—Here the plate fails by cracking along its greater axis; and since the deflection of a beam for a

given load is as the cube of the length, it is evident that the ends carry but a small part of the total load. Where the longer axis is more than twice the shorter one, we may neglect these end bearings entirely when we have the case of a flat plate supported at two opposite sides, which then becomes a simple beam: and this is the proper assumption to make in such a case. Making this assumption, and calling  $b$  the smaller dimension of the opening, we have

$$S = \frac{3}{4} \frac{wb^2}{t^2}, \quad \text{or} \quad t = \frac{b}{2} \sqrt{\frac{3w}{S}} \quad (40)$$

Prof. Bach gives for this case

$$S = \frac{Ca^2b^2w}{(a^2 + b^2)t^2} \quad (41)$$

where  $C$  is somewhere between  $\frac{2}{3}$  and 1.

When the longer axis is about  $1\frac{1}{2}$  times the shorter, as is common with manhole-covers, assume that  $\frac{3}{4}$  of the total load is carried at the sides, thus giving, from (41),

$$S = \frac{3}{4} \frac{3}{4} \frac{wb^2}{t^2}, \quad \text{or} \quad t = \frac{3b}{4} \sqrt{\frac{w}{S}} \quad (42)$$

## RESILIENCE

**33. Resilience Defined.**—Resilience is the work which a body can do in springing back after a deforming force has been removed. Within the elastic limit the work of the forces deforming the body equals the energy stored in the body, that is, it equals the resilience. If a body is stressed beyond its elastic limit some of the work of the forces is spent in permanently deforming the body through sliding of the particles over one another, thus causing a loss of heat. Under the latter condition the resilience equals that portion of the total work of deformation which the body can give back upon removal of the forces.

Since work is measured by the product of the force and the distance through which the application point moves in the direction of the force, we find the work of deformation by multiplying the average stress by the total deformation. Thus the work of deformation ( $W$ ) equals the resilience ( $K$ ) for any unit stress ( $S$ ) which is within the elastic limit and  $W = K = \frac{1}{2}Pe$ , where  $P$  is the maximum total stress and  $e$  the corresponding deformation.

If in the stress-deformation diagram for a body (Fig. 22) the total stresses ( $P$ ) are plotted as ordinates and the displacements ( $e$ ) as abscissæ, then the work of deformation ( $W$ ) equals the area (measured to scales of the diagram) between the curve and the  $e$ -axis. Within the elastic limit the resilience also is measured by this area. When the elastic limit has

1 experiment shows that the return curve is parallel to the  
ve— ' is parallel to  $oq$  Fig. 22. Therefore the resilience  
of the maximum stress multiplied by the corresponding  
na In Fig. 22 the shaded triangle marked resilience  
re nts. the resi-  
c to a maximum  
an elastic deformation  
corresponding total work  
forming stresses is measured  
area  $oq_2q_3d_3$  and the energy  
through friction by the area  
 $q_3'$ . The total work of deforma-  
to rupture is often called the  
gy of rupture.

If the permanent sets be laid off  
from the stress-axis opposite to the  
corresponding stresses— $p_2q_2''$  equal  
to  $oq_3'$ ,  $p_4q_4''$  equal to  $oq_4'$ , etc.—then  
it may also be shown that the  
following sets of areas are equal

$op_1q_1o = oq_1d_1o$ ,  $op_2q_2o = oq_2d_2o$ ,  $op_2q_2''q_3q_2o = q_3'q_3d_3q_3'$  and  $op_2q_2s''q_4''q_3q_2o = q_4'q_4d_4q_4'$ . Therefore the resilience equals the work of the elastic  
deformations.

There are three varieties of resilience commonly met: resilience of  
direct stress, tension or compression; resilience of cross-bending; and  
resilience of torsion. Values for these different kinds of resilience will now  
be determined. In all cases it will be noted that the resilience is directly  
proportional to the square of the maximum unit stress divided by the  
modulus of elasticity ( $\frac{S^2}{E}$ ), the volume  $AV$ , and a coefficient which  
depends upon the kind of stress, form of cross-section and the method  
of loading.

**34. Resilience of Bodies under Direct Stress.**—Consider a homogene-  
ous prism of uniform cross-section subjected to end axial pulls or pushes  
which are slowly increased until the value  $P$  is reached,  $\frac{P}{A}$  being less than  
the elastic limit. Then the total elongation due to  $P$  is  $e = \frac{Pl}{AE}$  (see eq.  
1) and the total work  $W_p$ , which is equal to the resilience  $K$ , is  $W_p = \frac{1}{2}Pe$ .  
Therefore

$$K = \frac{1}{2}SA \frac{SA l}{AE} = \frac{1}{2} \frac{S^2}{E} Al. \quad \dots \quad (43)$$

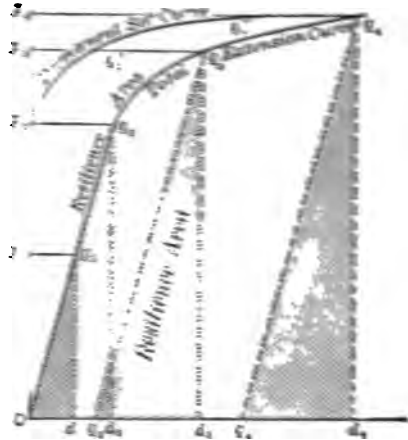


FIG. 22.

The factor  $\frac{1}{2} \frac{S_e^2}{E}$ , where  $S_e$  is the elastic limit, is the *modulus of resilience* for a material under the direct stress considered.

It should be noted in passing that when  $\frac{P}{A}$  exceeds the elastic limit, the resilience may be computed by eq. (43), but it does not equal the work of  $P$  in this case. The latter can be found from the area of the stress deformation diagram as previously indicated. It may also be approximated as follows:

For ductile materials having a stress-diagram like mild steel (Fig. 3), the energy of rupture  $K_r$  per unit of volume is approximately

$$K_r = \left( \frac{S_y + S_m}{2} \right) \epsilon_m. \quad . \quad . \quad . \quad . \quad . \quad (44)$$

For materials like cast iron having a parabolic stress-diagram, approximately,

$$K_r = \frac{2}{3} S_m \epsilon_m. \quad . \quad . \quad . \quad . \quad . \quad (45)$$

In the above  $S_y$  and  $S_m$  are the unit stresses at the yield point and maximum load, respectively, and  $\epsilon_m$  is the ultimate unit elongation.

**35. Resilience in Cross-bending.**—Consider a homogeneous cantilever beam of uniform cross-section with an end load which is gradually increased until it reaches the value  $P$ , the latter being of such value that the fibers of the beam are not stressed beyond the elastic limit. The deflection of the free end due to bending is  $f = \frac{Pl^3}{3EI}$  (eq. 27) and the work ( $W_p$ ) of the load is  $W_p = \frac{1}{2} Pf$ . Therefore the resilience  $K = \frac{1}{2} Pf = \frac{P^2 l^3}{6EI}$ . Since the maximum unit stress, on the extreme fiber at the support is  $S = \frac{Plc}{I}$  we have

$$K = \frac{S^2 Il}{6Ec^2} = \frac{S^2 A r^2 l}{6Ec^2} = \frac{1}{6} \frac{r^2 S^2}{c^2} Al. \quad . \quad . \quad . \quad . \quad (46)$$

Here  $r$  = radius of gyration and the other symbols are in accordance with previous notation. The result in eq. (46) also holds for a simple beam loaded at the middle.

Eq. (46) does not hold for stresses beyond the elastic limit because the flexure formula no longer gives the true value of  $S$ . The resilience for this case may be computed approximately from  $K = \frac{P_1^2 l^3}{6EI}$ , where  $P_1$  is the maximum load.

For beams with a uniform load the resilience may be gotten by

$$K = \frac{1}{2} \int w dx y^2 \quad \dots \quad (47)$$

where  $w$  is the load per unit of length,  $y$  is the deflection at any point and is expressed in terms of  $x$ , and the limits of integration include the entire beam.

1. Thus for a simple beam with a uniform load

$$K = \frac{1}{2} \int_0^l w dx \left[ \frac{wl^3x}{24} - \frac{wlx^3}{12} + \frac{wx^4}{12} \right] = \frac{w^2 l^5}{240EI}.$$

For the value of  $y$  in the above, see eq. (f), Art. 29. Since  $wl^2 = \frac{8SI}{c}$  the resilience in terms of the unit stress on the outside fiber at the center of the beam is

$$K = \frac{4}{15} \frac{r^2 S^2}{c^2 E} Al. \quad \dots \quad (48)$$

In all of these discussions the resilience due to shear has been neglected because of its small effect.

**36. Resilience in Torsion.**—Consider a homogeneous circular shaft held at one end and twisted by a couple, the magnitude of which increases to  $Pa$  without overstressing the shaft. Evidently the work of the couple is  $W_e = \frac{Pa\theta}{2}$ . Substituting in this equation  $\theta = \frac{Ml}{E_s J}$ ,  $M_t = \frac{S_s J}{c}$ , and  $J = Ar_0^2$ —where  $r_0$  = polar radius of gyration—we have

$$K = \frac{Pa\theta}{2} = \frac{1}{2} \frac{r_0^2 S_s^2}{c^2 E_s} Al. \quad \dots \quad (49)$$

For a solid round shaft  $K = \frac{1}{4} \frac{S_s^2}{E_s} Al$ .

**37. Resilience a Measure of Shock Resistance.**—The magnitude or effect of a blow, or of a falling body, is measured by the energy stored in the moving body at the instant of impact. In the case of a body which has fallen freely in space under the action of gravity, its energy is  $Wh$ , where  $W$  is the weight of the body, and  $h$  is the distance through which the body has fallen freely. In any translation, the energy of the body is  $\frac{Wv^2}{2g}$ , where  $v$  is the velocity, and  $g$  is the acceleration of gravity. If a moving body, as a falling weight, is stopped by striking a fixed solid body, the energy of the moving body is spent in one or all of the following ways:

\* An alternative method of computing the resilience for such cases is afforded by the equation  $K = \int \frac{M^2}{EI} dx$  where the variables are expressed in terms of  $x$  and the limits of integration cover the entire beam.



(a) In deforming the moving body itself, either within or beyond its elastic limit.

(b) In a local deformation of both bodies at the surface of contact, within or beyond the elastic limit.

(c) In moving the fixed body as a whole, with an accelerated velocity, the resistance consisting of the inertia of the body.

(d) In moving the fixed body against its external supports and resistances.

(e) Finally, in deforming the fixed body as a whole against the resisting stresses developed thereby.

If there is nearly absolute rigidity in all parts except in the body struck and if this yields only as a whole and not locally at the point of contact, then only can nearly all of the energy of the moving body be absorbed through deformation in the body struck. When the energy of the striking body is due principally to its mass (measured by  $\frac{W}{g}$ ) and only in small part to its velocity then 90 per cent or more of its energy may be absorbed by the body struck.

Assuming that all the energy of a blow is absorbed by the body struck,

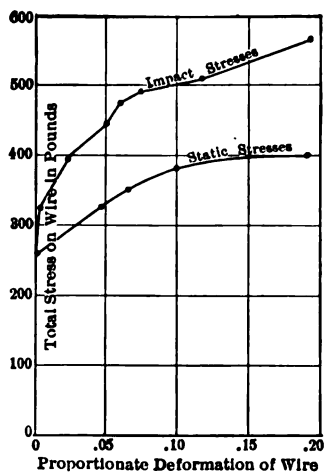


FIG. 23.—Comparison of Impact and Static Stresses when the Deformations are the Same. (*Report of French Com.*, Vol. 2, p. 344.)

then it becomes apparent that the work of deformation of that body must equal the energy of the blow. Studies of the relationships of resistance to deformation under impact, where the unit stresses have exceeded the elastic limit, have, in general, shown that a given deformation under impact is accompanied by a higher unit stress than in a static, or slowly applied, loading. For the tests on soft iron wire illustrated in Fig. 23, the work of deformation in impact, as measured by the area under the impact stress-deformation curve, is about 30 per cent greater than the corresponding value for static stresses. Likewise Russell \* found that the average energy of rupture of cast-iron bars tested in his pendulum impact machine (Fig. 18, Chapter II) was about 44 per cent greater than the energy of rupture under gradually applied loads. When the stresses are within the elastic limit it has

been customary to assume that the moduli of elasticity for static loadings hold for impact. It has also been customary to argue from the above

\* See *Trans. Am. Soc. Civ. Engr.*, Vol. 39, p. 246.

considerations that the modulus of resilience gotten from a static test is a measure of resistance to repeated shocks or blows and that the total area of the stress-diagram (the energy of rupture) is a measure of resistance to a single blow. In other words these quantities are considered indexes of toughness. That these conclusions do not apply, however, to all heat-treated materials is evinced by the behavior of burnt steel. This material often exhibits about as high energy of rupture in a static test as properly treated material but is far less tough. (See also Art. 711.)

A comprehensive study of the elastic portions of impact stress-deformation curves for our various building materials is badly needed in order that true value of the static modulus of resilience as an index of resistance to repeated shocks may be ascertained.

If we assume all of the energy of the blow is absorbed by the body struck and that the modulus of elasticity is independent of the speed of loading, we have for the case of direct stress under axial elastic impact

$$W(h+e) = \frac{1}{2} \frac{S^2}{E} Al. \quad . \quad . \quad . \quad . \quad . \quad . \quad (50)$$

Here  $W$  = weight falling,  $h$  = distance dropped,  $e$  = maximum deformation of body struck (the deformation will oscillate from a maximum to a minimum value as in a spring),  $S$  = maximum intensity of stress corresponding to deformation,  $E$  = modulus of elasticity,  $A$  = area of cross-section, and  $l$  = length.

Similarly for a beam under impact of a center load we shall have

$$W(h+f) = \frac{1}{6} \frac{r^2 S^2}{c^2 E} Al. \quad . \quad . \quad . \quad . \quad . \quad . \quad (51)$$

Here  $f$  = the maximum deflection and the other quantities have usual significance. When  $h$  in either of eqs. (51) or (52) is zero, namely when the load is suddenly applied, it is easy to show that the maximum intensity of stress is twice that for a static load,  $W$ .

It may also be shown that the deformation of a bar under axial elastic impact is given by  $e = e' + e' \left( 1 + \frac{2h}{e'} \right)^{1/2}$ , where  $e'$  is the elongation due to a static load  $W$  and  $e$  that due to the impact load  $W$ . Similarly for transverse impact we have  $f = f' + f' \left( 1 + \frac{2h}{f'} \right)^{1/2}$ , where  $f'$  is the deflection due to static  $W$  and  $f$  that due to impact of  $W$ .

MATERIALS UNDER COMBINED STRESS

38. Direct and Bending Stresses.—Consider the case of beam under a transverse bending moment  $M$  and subjected to an end load  $P$  lying in a principal axis, Fig. 24, for example. Consider

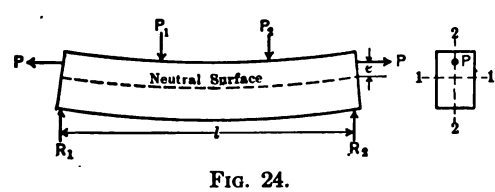


FIG. 24.

force  $P$  resolved into along the axis of the and a couple  $Pe$ . The unit stress on the external fiber at the danger section is given by

$$S = \frac{P}{A} \pm \frac{Mc}{I} \dots \dots \dots$$

Here  $M$  is the algebraic sum of the moments due to end loading ( $Pe$  transverse loadings taken on either side of the section,  $I$  is the moment of inertia of the cross-section about the axis 1-1,  $c$  the distance from the fiber considered,  $A$  the area of the cross-section. Unless the beam is long, deflection affects the moment arm of  $P$  but little and is, therefore, not considered in computing  $M$ .

Equation (52) may also be used in designing short columns ( $\frac{l}{r} < 10$ ) which are eccentrically loaded. It then reduces to

$$S = \frac{P}{A} \left( 1 \pm \frac{ce}{r^2} \right) \dots \dots \dots$$

Here  $r$  is the radius of gyration of the section about the principal axis which is normal to the lever arm of  $P$ .

If  $P$  does not lie on a principal axis eq. (53) does not hold. The stress on any fiber is then given by

$$S = \frac{P}{A} \left( 1 + \frac{xx_1}{I_y} + \frac{yy_1}{I_x} \right) \dots \dots \dots$$

Where  $x_1$  and  $y_1$  are the coordinates of the application point of the load with respect to the principal axes (the  $x$ -axis and the  $y$ -axes),  $x$  and  $y$  are the coordinates of the fiber with unit stress  $S$ ,  $I_x$  and  $I_y$  are the principal moments of inertia with respect to the  $x$  and  $y$  axes, respectively. Due account of the signs of  $x$ ,  $x_1$ ,  $y$  and  $y_1$  must be taken in using eq.

39. Shears and Direct Stress.—One of the most common cases of combined stress is direct stress with pairs of equal shears perpendicular and parallel to it. Let Fig. 25(a) represent the side elevation of an elementary rectangular parallelepiped, the thickness perpendicular to the paper being unity and the width and height  $dx$  and  $dy$ , respectively.

3. The test of the stress  $S_x$  and the strain  $\epsilon_x$  is made by the use of the strain gage. The strain gage is a device which measures the change in length of a material under stress. The strain gage is attached to the material and the change in length is measured by the change in resistance of the gage. The strain gage is a device which measures the change in length of a material under stress. The strain gage is attached to the material and the change in length is measured by the change in resistance of the gage.



FIG. 1

Resolving the stress  $S_x$  and  $S_y$  into components  $S_x \cos \theta$  and  $S_x \sin \theta$  and  $S_y \cos \theta$  and  $S_y \sin \theta$  respectively. The stress  $S_x$  is resolved into components  $S_x \cos \theta$  and  $S_x \sin \theta$  respectively. The stress  $S_y$  is resolved into components  $S_y \cos \theta$  and  $S_y \sin \theta$  respectively.

$$S_x \cos \theta = S_x \frac{x}{r} \quad (1)$$

$$S_x \sin \theta = S_x \frac{y}{r} \quad (2)$$

where  $r = \sqrt{x^2 + y^2}$  is the radius of the element.

where  $r = \sqrt{x^2 + y^2}$  is the radius of the element.

where  $r = \sqrt{x^2 + y^2}$  is the radius of the element.

$$S_x \cos \theta = S_x \frac{x}{r} \quad (3)$$

$$S_x \sin \theta = S_x \frac{y}{r} \quad (4)$$

where  $r = \sqrt{x^2 + y^2}$  is the radius of the element. The stress  $S_x$  is resolved into components  $S_x \cos \theta$  and  $S_x \sin \theta$  respectively. The stress  $S_y$  is resolved into components  $S_y \cos \theta$  and  $S_y \sin \theta$  respectively. The stress  $S_x$  is resolved into components  $S_x \cos \theta$  and  $S_x \sin \theta$  respectively. The stress  $S_y$  is resolved into components  $S_y \cos \theta$  and  $S_y \sin \theta$  respectively.

The unit deformation in the direction of max.  $S_n$  will be incre by the lateral deformation of min.  $S_n$  acting at right angles to the maximum value (see Art. 5 and 6) or  $\epsilon_n = \frac{\text{max. } S_n}{E} + \lambda \frac{\text{min. } S_n}{E}$ . Where  $\epsilon_n$  is the uni deformation along maximum  $S_n$  and  $\lambda$  is Poisson's ratio. Substituti values of  $S_n$  from eq. (58) and changing sign of min.  $S_n$ , there results

$$\epsilon_n = \frac{S}{2E}(1-\lambda) + (1+\lambda)\sqrt{S_s^2 + \left(\frac{S}{2}\right)^2} \quad (59)$$

If  $\lambda = \frac{1}{4}$ , a fair mean for metals, then

$$\epsilon_n = \frac{3S}{8E} + \frac{5}{4E}\sqrt{S_s^2 + \left(\frac{S}{2}\right)^2} \quad (60)$$

For analyzing the internal stresses in beams and shafting eq. (57) and (58) are much used. In beams the horizontal and vertical shearing stresses provide the  $S_s dx$  and  $S_s dy$  forces of Fig. 25 and the flexure stresses make up the  $S dy$  forces. In horizontal shafts the torsional shears in planes tangent to the shaft provide the pairs of shearing forces, and the stresses due to bending again introduce the normal forces. In vertical shafts the normal stress may be due to both direct stress (from the weight of the shaft) and lateral bending.

**40. Biaxial Loading.**—Fig. 26 shows an elementary parallelopiped

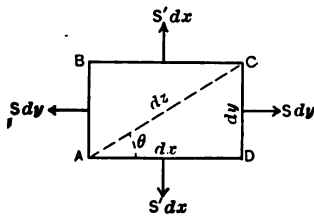


FIG. 26.

of unit thickness under two pairs of mutually perpendicular tensile stresses. The following analysis with proper regard to sign holds, in general, whether both pairs of stresses are of like kind or not. We wish to find the value of the normal ( $S_n$ ) and tangential ( $S_p$ ) unit stresses on any plane such as  $AC$ . Resolving  $S dy$  and  $S' dx$  parallel to  $AC$  and perpendicular to it and substituting  $\sin \theta$

$= \frac{dy}{dz}$  and  $\cos \theta = \frac{dx}{dz}$  there is obtained

$$S_p = \frac{S - S'}{2} \sin 2\theta, \quad (61)$$

$$S_n = S \sin^2 \theta + S' \cos^2 \theta. \quad (62)$$

When  $\theta = 45^\circ$   $S_p = \frac{S - S'}{2}$ . Note, however, that for this case the greatest value of  $S_p = \frac{S}{2}$ , if  $S > S'$ . This stress occurs in a plane parallel to  $S'$  and making an angle of  $45^\circ$  with  $S$ .  $S_n$  is a maximum when  $\theta = 90^\circ$  if  $S > S'$ ; when  $\theta = 0^\circ$  if  $S < S'$ .

E (61) and (62) are of value in determining stresses in boiler shells and pipes.

**41. Conditions Determining Elastic Break-down.**—There has always been much perplexity concerning what factors determine the breaking point of a metal at its elastic limit. Broadly speaking, the question is, when does failure begin when a certain maximum unit stress is reached, regardless of the way in which that stress is produced, or does it commence when unit deformation reaches a certain maximum. In studying these problems, experimenters \* have placed specimens of ductile metals under combined tension and torsion, compression and torsion, flexure and torsion, and under biaxial loadings. The three principal theories which have been advanced are as follows:

1. Elastic break-down begins when the maximum normal stress reaches a certain value—the *maximum stress theory*;
2. Elastic failure begins when the unit deformation ( $\epsilon_n$ ) in the direction of the maximum normal stress reaches a certain magnitude—the *maximum strain theory*;
3. Elastic break-down begins when the maximum shear ( $S_p$ ) reaches a certain magnitude—the *maximum shear theory*.

Most recent experimenters have held to the third theory, convinced that a ductile material under direct stresses, or direct and shear stresses, suffers elastic break-down when some plane in the body experiences a shear stress equal to the shearing elastic limit, and that failure ensues when the ultimate shear stress is imposed on some plane (see  $S_p$  of formula (57) and discussion in Art. 40). Becker,\* however, from a very careful set of tests on thin steel tubes under biaxial loading has shown that elastic break-down will ensue if the maximum unit strain exceeds a certain value, also if the maximum unit shear stress exceeds the shearing elastic limit. In other words, for biaxial loading he found that as the ratio of  $\frac{S'}{S}$  was increased the elastic limit was reached as soon as the maximum unit strain reached a certain value; when, however, as  $\frac{S'}{S}$  increased, the maximum  $S_p$  ( $=\frac{S}{2}$  if  $S > S'$ ) reached the shearing elastic limit before the maximum unit strain  $\epsilon_n$  reached its critical value, then failure was conditioned by the maximum shear theory. The two limiting conditions for biaxial tensions are given by  $\epsilon_n' = \frac{S}{E} - \lambda \frac{S'}{E}$  and  $S_p' = \frac{1}{2}S$ , where  $\epsilon_n'$  = maximum unit elastic deformation which material can withstand and  $S_p'$  = shearing elastic limit.

\* See accounts of researches by J. J. Guest, *Phil. Mag.*, July, 1900; W. A. Scoble, *Phil. Mag.*, Vol. 12, 1906, p. 583; E. S. Hancock, *Proc. A.S.T.M.*, Vol. 5, p. 179; Vol. 6, p. 295; A. J. Becker, *Bulletin No. 85*, Univ. of Illinois Expt. Sta. Also see Art. 726.

That the theory advanced by Becker holds for brittle materials can be asserted in the present state of our knowledge.

For the present it seems safe in designing or investigating members under combined stresses to determine both maximum  $S_n$  and  $S_p$ , see that neither exceeds the respective allowable working unit stress for the material.

## CHAPTER II

### MACHINES AND APPLIANCES FOR MECHANICAL TESTS

#### TESTING MACHINES

**42. Definition.**—Since it would be obviously impossible to break, or even deform considerably large specimens of strong material by the imposition of dead weights, a mechanical device called a testing machine is employed for this purpose. A testing machine may be said to consist of a base or frame which supports the crossheads used in loading the specimen, a means for applying the load, and a device for measuring it.

#### UNIVERSAL TESTING MACHINES

**43. Classes of Universal Testing Machines.**—Universal testing machines—those in which tension, compression or cross-bending tests can be made—are provided with at least one fixed crosshead against which the specimen rests and a movable crosshead by means of which the specimen is deformed. In accordance with the method employed to drive the movable crosshead, such machines may be divided into two classes, hydraulic machines and screw-gear machines. In American laboratories most of the testing machines of less than 600,000-lb. capacity are of the latter type. Many of the larger testing machines, however, are of the hydraulic type.

With the exception of the Emery testing machine, which belongs in a class by itself, the chief advantages of hydraulic testing machines are freedom from vibration and noise, cheapness, and simplicity of construction and operation. On the other hand, most of the testing machines of this type have one or more of the following disadvantages: leakage of fluid, variable friction at the stuffing boxes and around the ram, or inaccurate means of measuring fluid pressures.\*

In American screw-gear testing machines the load is generally reduced by a system of compound levers and weighed by balancing with a poise which is moved along a graduated scalebeam. In general these machines

\* The University of Wisconsin has a 600,000-lb. universal hydraulic testing machine designed and erected by the instructional staff. As a result of calibrations the probable error of results gotten from this machine is not over 2 or 3 per cent for loads above 50,000 lb. The machine cost \$3500. (See *Proc. of Am. Soc. for Testing Materials*, Vol. 10, p. 551.)



may be made very sensitive and accurate, but when built in large capacities are more expensive than those of the hydraulic type. Owing to the vibration and noise produced by the driving mechanism employed in screw-gear machines, hand power is necessitated when very sensitive apparatus is used to measure deformations.

Testing machines are often called horizontal or vertical in accordance with the position in which a tension or compression specimen is held. The advantage of the former type lies in the accessibility of all parts of machine and specimen. In vertical testing machines, however, lateral bending due to the weight of the specimen is obviated.

**44: General Conditions which Should Obtain in Universal Machines.—**

The following considerations apply to testing machines in general:

1. The weighing apparatus should be quite independent of the loading apparatus, the former usually being fixed and the latter movable.

2. In lever machines the length of the knife-edges must be proportioned to the maximum loads in order not to be crushed down, and they should be so placed that all will receive their share of the load. They must also be so mounted as not to change the leverage by any reaction displacement which may occur. To insure this, the knife-edges must be attached to the levers, and the bearings to the platform. Clearance between knife-edge bearings and levers must be sufficient to insure against frictional resistances, which greatly impair sensitiveness.

3. The knife-edges and bearings of any beam must lie in the same straight line, and this line should lie in the gravity axis of the beam and its rigid attachments. This is especially necessary for the weighing-beam itself, so that its vertical angular movement may not disturb the counter-balancing. If the poise is moved by a cord over a pair of pulleys, this cord should be attached to the poise-hanger in this same axial line, so that the pulling of the poise may not supply a leverage on the beam to raise or lower it.

4. Manometer machines have many peculiar errors. For example, any air-bubble in the indicating liquid vitiates the results by its own change in volume under pressure. Again, the exact area of surface subjected to pressure is always uncertain.

5. The weighing apparatus should be so constructed as to be readily verified by the imposition of known weights, and the parts should be open to inspection and easily repaired and kept in order.

6. A precision of 1 in 250 has been considered sufficient.\* This is a proportional error of 0.4 of 1 per cent. Also, the imposition of a load equal to one-two-hundred-fiftieth of that on the machine should produce an appreciable indication on the weighing device.

\* This standard is given by the French Commission and has been accepted by the Am. Soc. for Testing Materials (see *Standards*, 1916).

7. The loading should proceed gradually and uniformly, and not by sudden increments as by large pump-pulsations, or by the adding of weights by hand to the weighing-beam. The rate of loading should be under perfect control.

8. The machine should be so constructed as to permit the free use of appliances for measuring distortion of the specimen by some suitable device.

9. For compression tests the speeds of the pulling head should not be over one-tenth or one-twentieth of an inch per min.

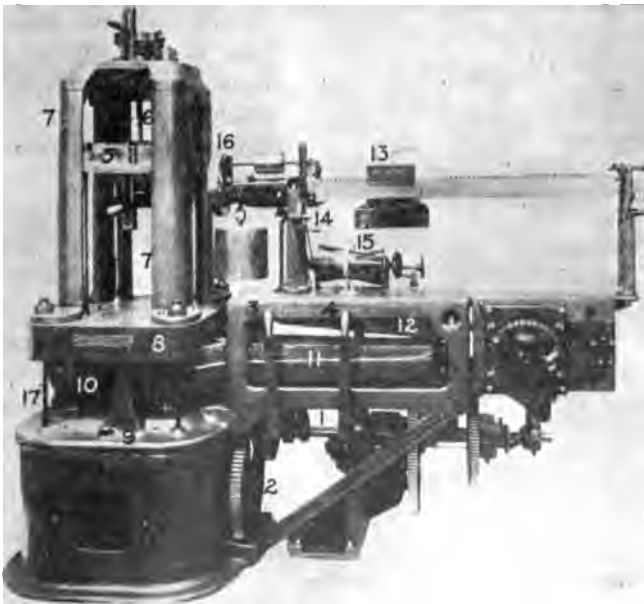


FIG. 1.—Olsen's Automatic Universal Testing Machine with Direct Motor Drive.

10. The axes of the jaws in the crossheads of the machine should be so placed that they will remain in line throughout any test provided that the specimen is properly centered in them.

11. There must be no twisting or rocking of the movable head when in motion.

**45. Olsen Testing Machines.**—The Olsen screw-gear machine shown in Fig. 1, affords a good illustration of the universal machines common to American laboratories. Machines similar to this are built in capacities from 30,000 to 400,000 lb.; others embodying similar principles are built in capacities up to 1,000,000 lb. Power is applied to the machine illustrated through a direct connected motor attached in the rear as shown.

The main shaft (1) transmits power to the gear (2) which, through shaft and system of gearing, rotates four straining nuts. Each of the straining nuts bears against the underside of the bed plate of the machine (9) and revolves about one of the four vertical screws which are rigidly fastened to the pulling head (5). In this manner an upward or downward translation may be imparted to the screws and pulling head.



FIG. 2.—A 10,000-lb. Wire-testing Machine.

A wide range in the rate of motion of the pulling head may be secured by means of the clutch levers (3) and (4). The pull on the specimen (6), which is held in the jaws of the cross-heads by means of grooved wedges or liners, is transmitted downward through the four cast-iron columns (7) to the weighing table (8). The reaction of the straining nuts on the lower end of the screws produces an equal upward force beneath the bed plate (9). The weighing table is supported on knife-edges fastened in the compound levers (10) and (11), which in turn are pivoted on pedestals formed on the bed plate (9); a closed circuit of forces is thus produced. The force on the weighing table is reduced by the levers (10), (11), (12) and balanced by the poise (13) on the scalebeam. The poise (13) is driven by a screw running along the top of the scalebeam. The operator may drive the poise by hand through the wheel (14) or the

motion of the poise may be automatically controlled by the variable speed cone drive (15) which is connected to the driving mechanism on the machine and drives the poise to the right as long as the scalebeam is up. When the beam drops an electric circuit is broken and the poise stops. The load in thousand-pound units or multiples thereof may be gotten from the graduations on the scale beam. Intermediate values of the load are read on the dial (16) which is driven by

the screw actuating the poise. When a specimen breaks some means must be provided for dissipating the stored energy or serious injury may be done to the knife-edges. Such dissipation of energy is often accomplished by providing long bolts equipped with cushion-seated nuts called recoil buffers (17).

Compression tests are made by attaching a compression-block to the lower side of the movable crosshead, and inserting the specimen between it and the weighing table (8).

The machine is arranged for cross-breaking tests by placing end bearings on the weighing table (or on an I-beam resting on this table

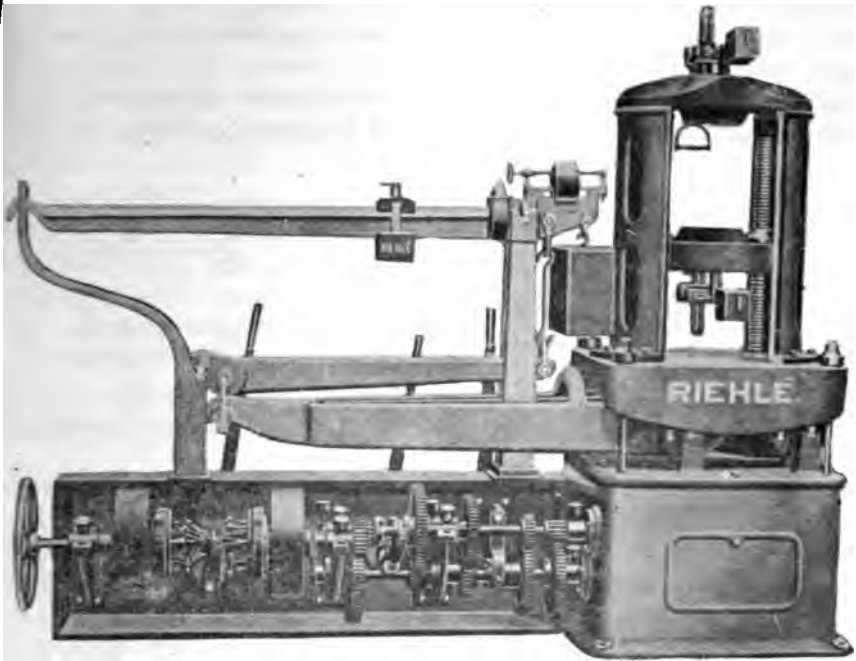


FIG. 3.—A Two-screw Universal Testing Machine.

if the specimen is long), and attaching a knife-edge bearing to the under side of the moving crosshead.

For testing small specimens hand power machines of the type shown in Fig. 2 have proven very serviceable. These machines are made in capacities ranging from 10,000 to 20,000 lb.

**46. Riehle Testing Machines.**—An equally well-known American type of testing machine is that manufactured by Riehle Bros. Fig. 3 illustrates a two-screw, 100,000-lb. automatic and autographic Riehle universal testing machine. The main difference in Olsen and Riehle testing machines lies in the method of moving the pulling head. In the Riehle

type the screws revolve in nuts in the pulling head. The main advantage claimed for a two-screw machine is the accessibility of the specimen. This advantage is partially offset by the care which must be exercised in centering compression or transverse specimens to avoid bending the screws.

**47. The Emery Testing Machine.**—The Emery method of weighing loads is generally considered the most sensitive yet devised.

The essential principle of this machine consists in a means of transmitting a definite percentage of the force applied to the specimen to the scale beams, and there weighing it accurately, without any friction whatever in the receiving, transmitting, or weighing parts. Hence a very small increment of the force applied is weighed with equal accuracy, whether this increment is added to a great or to a small previous load.\* This is accomplished by means of an hydraulic leverage and through the use of steel plate fulcrum instead of knife-edges. Thus, in Fig.

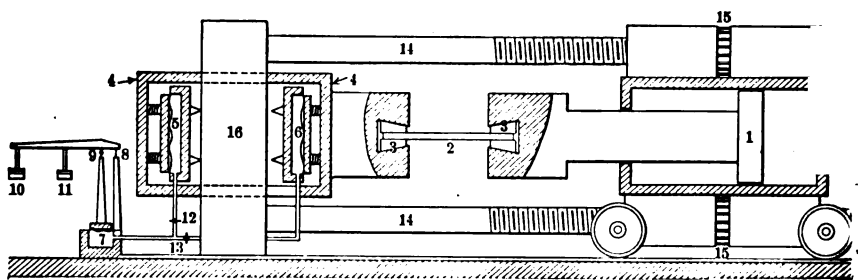


FIG. 4.—A Schematic Drawing Showing a Side Elevation of an Emery Testing Machine.

which is merely a schematic drawing, the load from the ram (1) is transmitted through the specimen (2) to the yoke (4), which rests against the hydraulic support (5). A pressure of equal intensity to that produced upon the liquid in the hydraulic support is transmitted to the hydraulic chamber. In principle, an hydraulic support or chamber consists of a very strong shallow metal cylinder covered by a thin diaphragm or of a plate with a thin diaphragm around the edge. The diaphragm permits a slight movement of the cover without leakage of the fluid. The total force acting against the top of the chamber (7) is to the force in the specimen as the effective area of the chamber top is to the effective area of the hydraulic support. The force on the chamber top is balanced by levers articulating elastically about thin steel plate fulcrum (8).

\* When the first machine was tested, a steel bar 5 in. in diameter was first broken under a load of 722,800 lb., and then a single horse-hair was tested, and the machine gave the strength (16 oz.) of this as accurately as a small spring-balance which was used for a check. *Rep. U. S. Test Board, Vol. 2, p. 1.*

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imum available height is 8 in. Rapid adjustment of the ram is secured by the lever at the left. The extensible lever at the right



FIG. 5.—A 30-ton Hydraulic Press  
Crushing Small Cubes of Concrete.



FIG. 6.—The Largest Testing Machine in the World. Located at the  
Laboratory of the U. S. Bureau of Standards.

used to load the specimen. The total weight of the press is Presses of this type are especially adapted for work in the field.

**The World's Largest Testing Machine.**—In Fig. 6 is shown the 2,000-lb. compression machine built by Tinius Olsen & Co, for the Surg Laboratory of the U. S. Bureau of Standards. This machine in present condition can be used to test specimens less than 25 ft. and 4 by 4 ft. square within the capacity of the machine. The screws are each  $13\frac{1}{2}$  in. in diameter. Connected to the screws by lower-driven nuts is the adjustable upper head, weighing 30 tons. A spherical-seated lower head of the same weight is mounted upon end of the ram of a huge hydraulic press, not shown in the illustration. To obviate uncertainty regarding frictional forces, the packing used in this ram is so designed that these forces will vary directly with the pressure, a condition for which due allowance is made in the calculation. The intensity of the fluid pressure in the ram is transferred to a smaller hydraulic chamber which is sealed with a diaphragm and connected on the pump well. The pressure on the diaphragm is increased through the lever system and screw-driven poise. The scale-ram is graduated to read loads on the specimen up to 2,000,000 lb. directly, and with end weights up to the capacity of the machine. Fluid pressure is supplied to the ram by means of a triple-plunger variable-stroke pump driven by a 15-h.p. variable-speed motor. This combination makes possible a considerable range in the speed of the lower head of the testing machine when desired. An air reservoir inserted in the supply line leading from the pump to the ram reduces the effects of the pulsations of the pump.\*

#### TRANSVERSE TESTING MACHINES

**51. General Remarks on Transverse Testing Machines.**—On account of the large amount of testing done upon beams, it is often convenient to have machines especially adapted for transverse testing.

The main conditions, other than those previously mentioned in Art. 44, which should be fulfilled by a transverse testing machine are:

1. The parts of the machine should be sufficiently rigid so that the tops of the knife-edges supporting the beam and the bottom of the loading knife-edge will remain perpendicular to the sides of the beam throughout the test.

2. The supporting and loading knife-edges should, if necessary, bear upon auxiliary plates to prevent indentation. Also, these knife-edges should be so arranged that longitudinal tension in the specimen due to their rigidity is obviated. (See Art. 74.)

3. The machine should be equipped with a variety of slow speeds by which loads may be uniformly applied to the specimen at a rate which will permit the necessary observations to be made.

\* This machine is described in the *Eng. Record*, Sept. 28, 1912, p. 353.



**Descriptions of Various Transverse Testing Machines.**—  
 A 100,000-lb. Riehle hydraulic transverse testing machine.  
 The valve is operated by the hand wheel at the center of the

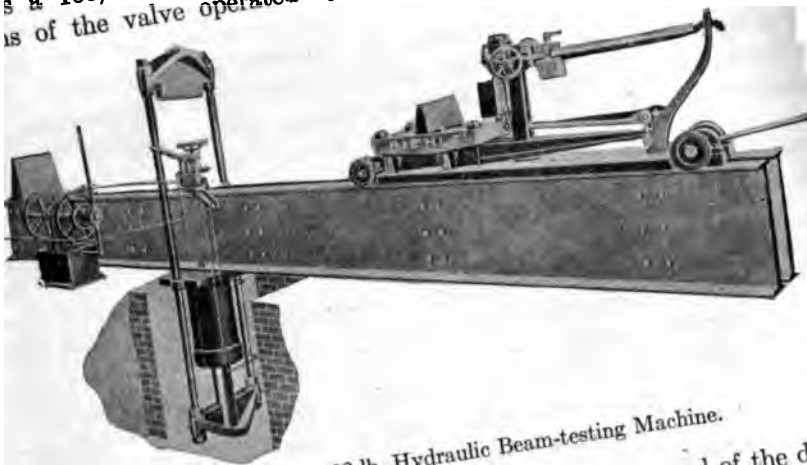


FIG. 7.—A 100,000-lb. Hydraulic Beam-testing Machine.

machine fluid from the pump can be forced into either end of the double-acting ram, which moves the loading knife-edge. The support at the left end and the weighing apparatus at the right may be adjusted to spans varying from 4 to 20 ft. To conserve space it will be found advan-

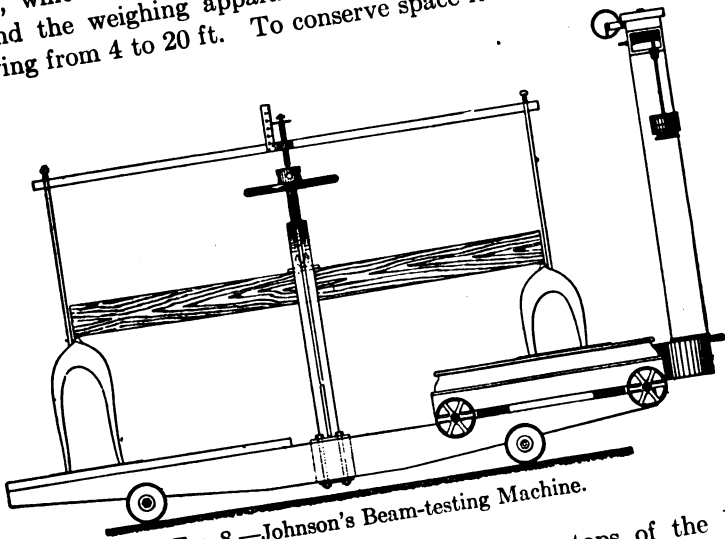


FIG. 8.—Johnson's Beam-testing Machine.

tageous to install this type of machine with the tops of the I-flush with the floor.

In Fig. 8 is shown a convenient device for testing small tin concrete beams. The micrometer-screw deflectometer reading t

l to measure the movement of the center bearing with respect end supports. If due care is taken to prevent crushing at the faces, the deflection will be equal to the movement registered micrometer-screw.

her form of transverse testing machine, much used in foundry is that shown in Fig. 9. In this machine the deflection is indicated on the graduated arc by means of an ingenious lever system not

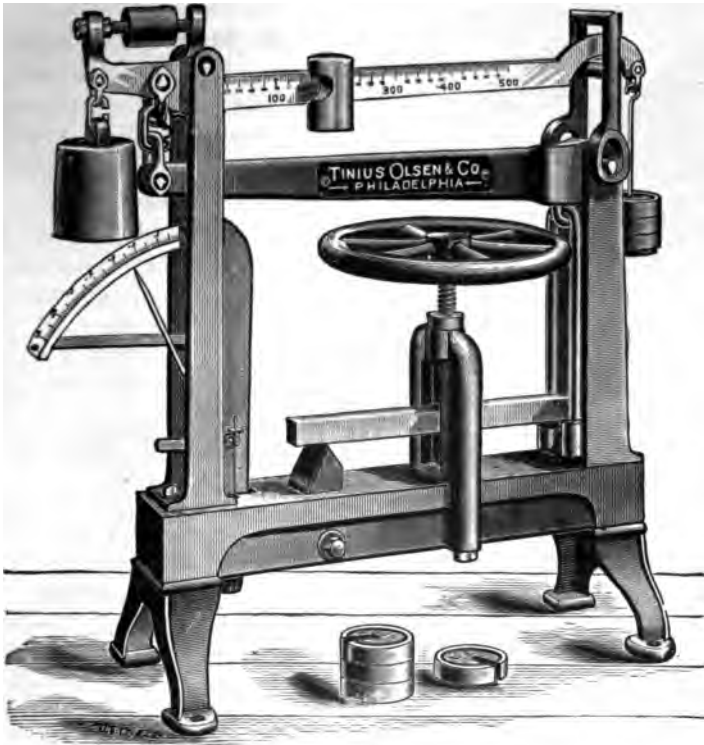


FIG. 9.—Cross-bending Testing-machine for Cast Iron. Deflection correctly measured.

shown in the figure. The capacity is 5000 lb., and the length of span 1 ft.

Fig. 10 shows a transverse testing machine which records the load-deflection curve upon a specially ruled diagram. Imposition of the load is made through the hand wheel at the top. The load on the specimen actuates a system of levers which move the heavy pendulum along the graduated arc, shown beneath the frame. This motion produces a rotation of the drum. Through a follow pin placed at the center of the span directly under the specimen, a record of the deflection is gotten.

The movement of this pin multiplied twenty times is transmitted to the pencil which moves vertically. Thus abscissas on the diagram measure loads and ordinates deflections. The machine is built for 12- or 24-in. spans. The capacity is 5000 lb.



FIG. 10.—Olsen's Autographic Cross-bending Machine.

by blows. The latter method is subject to much wider variations than the pressure method, but properly made is a more severe test than the former.

**54. A Cold-bend Attachment.**—A bending attachment made by Riehle Bros. for use in universal testing machines is shown in Fig. 11. The upper tool, which is clamped in the pulling head of the machine, carries the pin around which the specimen is to be bent. The lower tool is supported on the weighing table and is provided with two adjustable brackets. Before performing a test these brackets are properly spaced to allow clearance for the pin and twice the depth of the specimen. Pins of different sizes are provided with the attachment. The appliance illustrated will bend bars up to  $1\frac{1}{4}$  in. square through 180 deg.

#### COLD-BEND TESTING MACHINES

**53. Methods of Making the Test.**—Up to this time, only machines in which the bending is accomplished by static pressure have been especially made for cold-bend tests. The power hammer is ordinarily employed when bending is accomplished



FIG. 11.—A Bending Attachment.

**55. Olsen's Cold-bending Testing Machine.**—Fig. 12 represents a motor-driven cold-bend testing machine which will bend rods up to 2 in. in diameter or plates less than 6 in. wide. The specimen is held firmly by a wedge behind the left end, an adjustable vertical center pin, and an adjustable vertical pin placed back of the right end. The latter pin bends the specimen. By means of graduations on the table of the machine any bend up to 180 deg. may be determined. The wire grating prevents injury to the operator through a sudden break of the specimen, but permits the making of observations during testing.

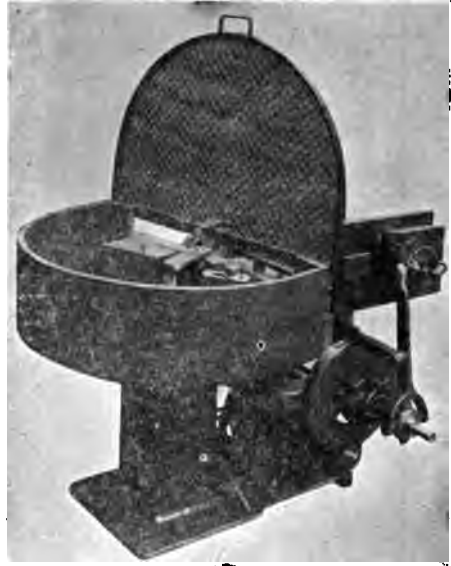


FIG. 12.—Olsen's Improved Cold-bending Machine.

#### SHEAR AND TORSION TESTING MACHINES

##### **56. Transverse Shear Test Appliances.**

—Although it is impossible to produce in a body a transverse shearing stress unaccompanied by tension or compression, yet, on account of the frequent occurrence of shearing stresses in members subjected to cross-bending—in rivets, bolts, pins, etc.—and on account of the weakness of timber in shearing along the grain, it is often desirable to obtain an approximate knowledge of the shearing strength of a material. Several devices have been employed for this purpose; two will be briefly described.

In Fig. 13 is shown a double shear apparatus designed to determine the shearing strength of metals. With this appliance the end and central portions of the specimen are tightly gripped by hardened steel bearing plates which minimize bending distortion. However, on account of the small clearance between the central and end portions of the device, some frictional resistance is developed in testing. A comparison of the results of shearing tests on this apparatus with values gotten from torsion tests of very thin cylinders of like material indicates that the shearing device gives about 10 per cent higher unit stresses. The bars used in these tests were uniform in size. Grooving the bars in the planes of shearing would undoubtedly have lessened the error.

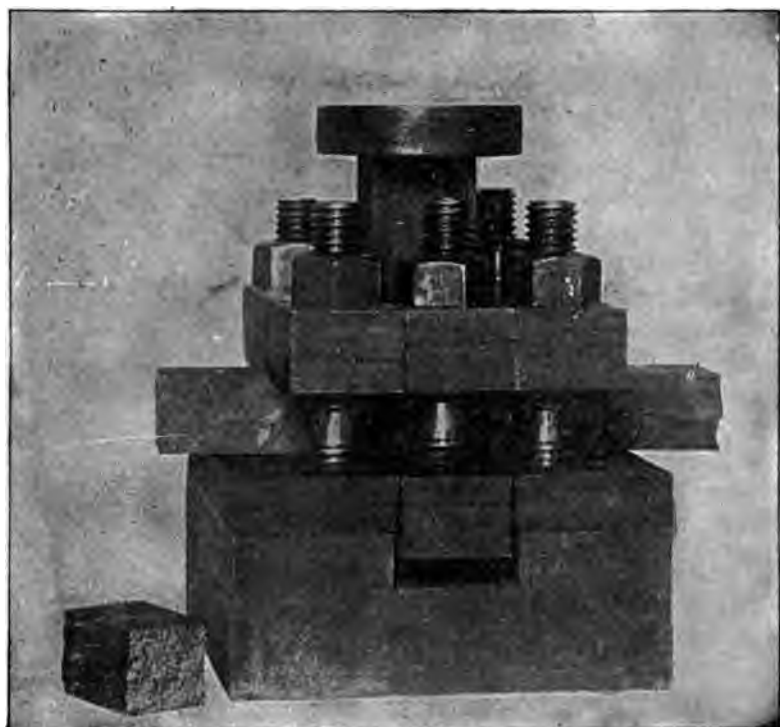


FIG. 13.—Johnson's Shear Tool.

A simple and satisfactory apparatus for making shearing tests on timber is shown in Fig. 14.

**57. Torsion Testing Machines.**—Shafting, wire, and elements of machines which are to be subjected to twisting are often tested in torsion to determine their torsional strength and shearing modulus of elasticity. Fig. 15 shows an Olsen 230,000-lb. torsion testing machine. This machine will take shafting up to  $2\frac{1}{2}$  in. in diameter and 22 ft. long. The motor operating through a chain drive and train of gears rotates a large gear carrying the chuck into which the right end of the specimen is securely fastened. The left end of the shaft is gripped by a similar chuck and transmits the twist to the lever system in the weighing head. To allow for longitudinal deformation of the specimen under test, the main lever of the weighing system is mounted on rollers, thus permitting considerable horizontal motion of the chuck. The lever shown at the left is employed to adjust the position of the weighing head on the rails.

A very perfect machine for testing wires from 0.05 in. to 0.18 in. in diameter (No. 18 to No. 7 B. W. G.) and for giving (a) the breaking moment, (b) the number of turns, and (c) the complete stress-diagram,

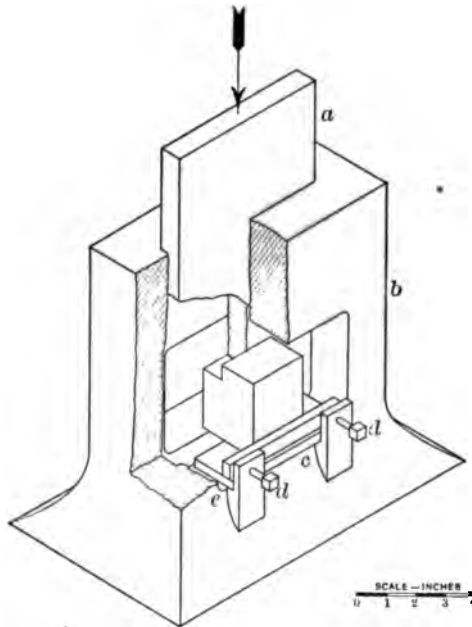


FIG. 14.—Shear Test Apparatus Used at the Forest Products Laboratory. Specimen is cut with a projecting shoulder. (Paper by Betts and Greeley at *Int. Engr. Congress*, 1915.)

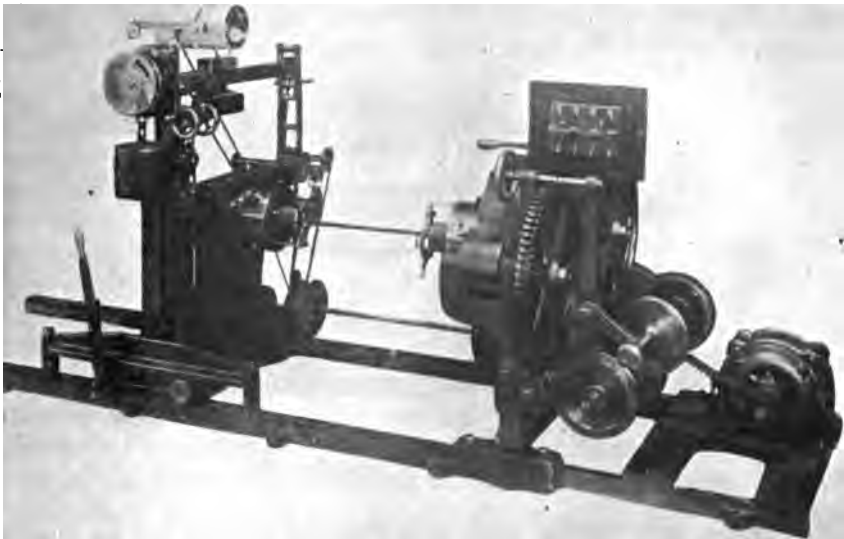


FIG. 15.—Olsen's Improved Torsion Testing Machine.

is shown in Fig. 16.\* This machine was used by Prof. Tetmajer and described by him in Vol. 4 of his *Communications*. The specimen is kept in tension during the test by a weight suspended by a cord con-

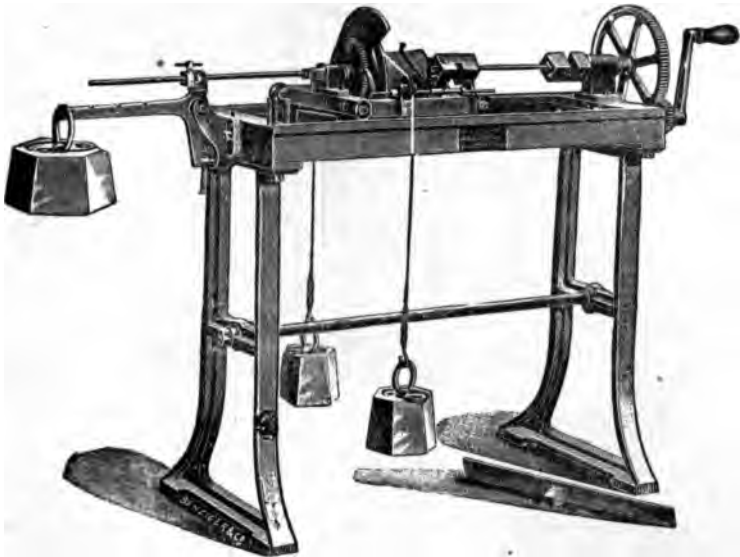


FIG. 16.—Tetmajer's Torsion-testing Machine for Wires, giving Autographic Records.

nected to the carriage at the resisting (and recording) end of the specimen. The resisting moment is developed by means of two weights suspended by cords which run in symmetrically arranged spiral grooves.

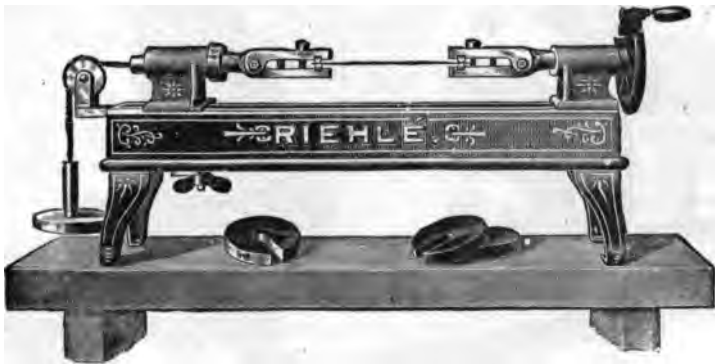


FIG. 17.—A Riehle Wire Twister.

A simple wire twisting machine without recording apparatus is shown in Fig. 17.

\* Made by Messrs. Amsler-Laffon & Sons, Schaffhausen, Switzerland.

## IMPACT TESTING MACHINES

**58. Essential Conditions for Impact Testing Machines.**—Since impact tests are made upon car wheels, car-couplers, car-axes, rails and rail joints to determine the resistance of such structural forms to shock, it is very necessary that the machine in which the tests are made shall be so constructed that the energy absorbed by the specimen can be determined. Inasmuch as the effect of a blow on the specimen depends upon the resistance of the specimen as well as the energy and mass of the falling body, it follows that a standard impact test involves a standard anvil resting upon a standard foundation, a standard hammer or tup, and a standard fall. Furthermore, in drop machines the axes of the tup, guides, anvil and specimen must be collinear; and the faces of the specimen, anvil, and tup must be parallel. On account of the impossibility of providing either an absolutely rigid anvil or tup, true quantitative results cannot be gotten. However, qualitative results of great value may be gotten from standardized tests. In practically all impact testing machines a transverse blow is delivered to the specimen either by a pendulum revolving about a horizontal axis or by a falling weight.

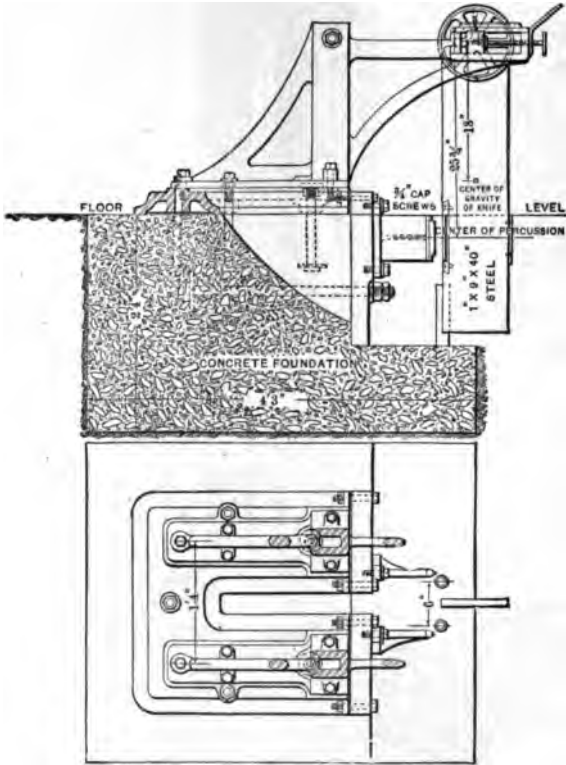


FIG. 18.—Russel's Impact Testing Machine.

**59. A Pendulum Impact Testing Machine.**—A good representative of this type of impact machine is shown in Fig. 18. The two knife-edge supports for the specimen are separated by a free passage for the pendulum, so that by raising higher than necessary to break the specimen on the first blow, the energy left in the pendulum carries it past the



vertical position. By registering, or observing in any suitable manner, the final forward position of the pendulum after passing the broken specimen the residual energy left over after rupture becomes known. This, subtracted from the potential energy due to the initial position of the pendulum, after making due allowance for the working resistance to

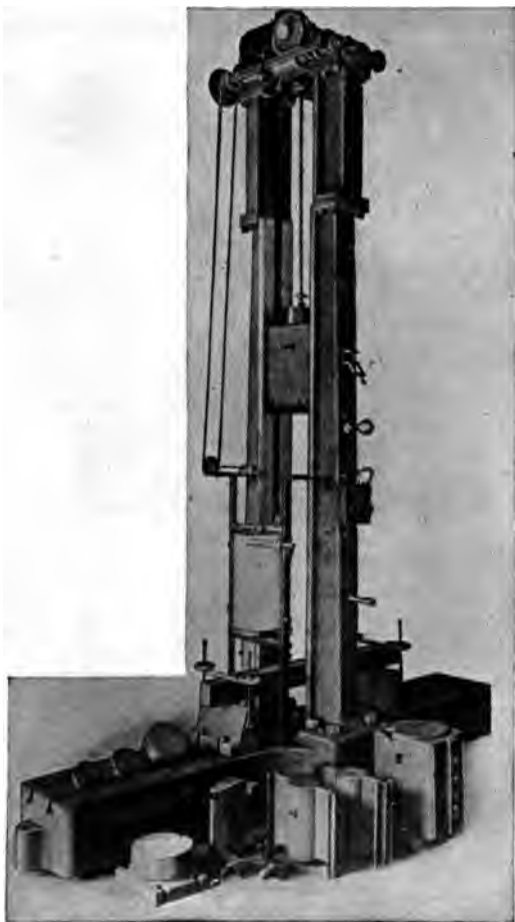


FIG. 19.—A Turner Impact Testing Machine.

motion, leaves the amount of energy absorbed by the specimen. The specimen supports are massive, and are firmly bedded upon a large body of concrete, so that they are very rigid. The center of percussion of the pendulum is carefully determined and the specimen placed at this point with the pendulum hanging vertically, the pivot-blocks being adjustable. The working resistance to motion is made as small as possible. The pendulum itself should rest on knife-edge bearings and should be a heavy flat bar, swinging edgewise, as here shown, to reduce the air resistance. The registering apparatus necessarily has some resistance, but all these can be evaluated by swinging the pendulum freely and noting the loss of energy for a single passage.

**60. Drop Impact Testing Machines.**—Fig. 19 illustrates a Turner impact

testing machine for either compression or transverse impact tests. In this machine the tup is hoisted by an electric motor placed at the top. Release of the tup is secured by reversing the current through the suspending magnet. A pencil moving over paper fastened to the revolving drum, shown just above the specimen, records the behavior of the specimen. Two sizes of this machine are made, the larger having



a capacity for beams 8 ft. long and 12 in. wide; the maximum drop attainable is 6 ft., and the weights of tup range from 50 to 500 lb. With this type of machine tests can be made to ascertain the elastic properties in addition to the energy of rupture of a specimen.

In Fig. 20 is presented a drawing of the standard drop testing machine adopted by the American Railway Engineering Association for making tests on rails. A noteworthy fact about this machine is the method of supporting the anvil. The latter weighs 20,000 lb., and is supported on 20 M. C. B. A. Standard "C" springs arranged in groups of five at each corner of the anvil. No connection exists between the vertical guides and the anvil. For further information relating to this machine see *The Manual* of the A. R. E. A.

#### APPARATUS FOR DETERMINING HARDNESS

**61. The Sclerometer.**—The scratch test has long been in use by the mineralogist to compare the relative hardness of minerals.\* On account of the variability of the mineralogist's standard it is not satisfactory for the user of metals. Turner's sclerometer or a modification of it has often been employed by technologists to abrade metals. In this device a diamond point is fixed at the base of a vertical pencil which is carried by a perfectly balanced arm. Provision is made for loading and moving the arm. The weight required to cause the diamond point to visibly scratch the surface of the specimen is taken as an indication of the hardness. Martens in using this apparatus determined hardness by measuring the width of scratch made under a constant load. The method is open to objection on account of the impossibility of grinding the diamond to a standard form.

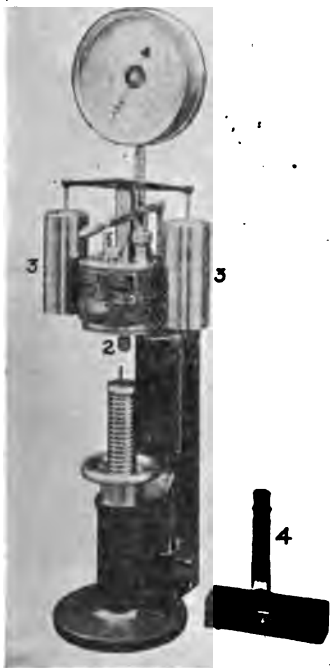


FIG. 21.—Olsen's Apparatus for Making the Brinell Hardness Test.

#### **62. The Brinell Ball Indentation Test.**

—A great many methods of determining the resistance of materials to indentation have been tried. Among the more noteworthy of these may be mentioned the punch used by Rodman, the knife-edge employed by Unwin, the cone

\* See Mohs' scale in Art. 252.

adopted by Ludwik, the ball perfected by Brinell, and the scleroscope invented by Shore.

In Fig. 21 is shown a common form of apparatus used by the followers of Brinell. The specimen is first made smooth and plane at the impression spot. It is then placed on the table (1) and raised until it touches the hardened steel ball (2). The latter is hung on the piston of an hydraulic ram which is used to load the specimen. In order to secure an accurate indication of the test pressure, the ram is provided with an accumulator loaded with dead weights (3). The latter rise when the desired pressure has been imposed upon the test-piece. Fig. 22 shows an inexpensive apparatus for making the test which may be inserted in any compression testing machine.

The ball generally used in this test is 10 mm. in diameter, and the pressure most frequently employed for iron or steel is 3000 kg. Brinell determines hardness upon the following basis. If the load in kilograms is divided by the area of the indented surface expressed in square millimeters, the average intensity of the pressure is determined. These intensities are called Brinell's hardness numbers. In determining the indentation the common practice has been to measure the impression left in the specimen by means of an especially constructed microscope (4). A much better method adopted by Devries of the U. S. Bureau of Standards follows.

By means of a microscope which is sighted upon a mark on the plunger the operator measures the indentation under a very light initial load, applies the test pressure, and again reads the indentation under the initial load. The difference between initial and final readings gives the correct indentation. Devries also found that a linear relation existed between load and depth of indentation for pressures less than 3000 kg.\*

**63. The Shore Scleroscope.**†—In this apparatus the specimen is struck by a small diamond-pointed hammer falling freely from a height of about 10 in. The rebound of the hammer is measured and gives an indication, according to its inventor, of the resistance to indentation or hardness. Fig. 23 shows the scleroscope. The essential features of the instrument are an accurately ground vertical glass tube placed in front of a scale graduated into 140 equal parts, a diamond-pointed hammer within the tube, and a vertical rod which is employed to plumb the apparatus. The hammer can be raised and suspended at the top of tube by producing a partial vacuum with the bulb. By again squeezing

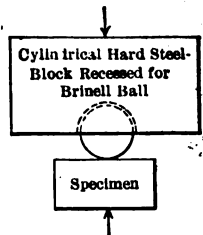


FIG. 22.—Auxiliary Device for Making the Brinell Test in a Universal Testing Machine.

\* *Proc. Am. Soc. for Test. Materials*, Vol. 11. 1911, p. 709.

† Manufactured by Shore Instrument and Mfg. Co., 267 24th St., N. Y.

the bulb a sufficient amount of air is admitted to the tube to release the hammer without giving it initial velocity. The height to which the hammer rises after striking the specimen is noted by the eye. Since slight differences in the sharpness of hammer points greatly affect the rebound, the hammer must be frequently tested on a standard surface. The surface ordinarily used is of hardened steel which gives a rebound of 100 on the scale.



FIG. 23.—Shore's Scleroscope.

#### ENDURANCE TESTING MACHINES

**64. Wöhler's Repeated-stress Testing Machines.**—Owing to value and importance of the systematic studies of Wöhler upon the fatigue of metals, his appliances have received considerable attention. Fig. 24 shows his apparatus for repeated tensile stresses. Here the specimen (*A*) is stressed through the lever (*L*) and spring (*s*) acting on the auxiliary lever (*m*). The pull of the spring (*s*) is measured by the starting of the adjusted calibrated spring (*s'*) through the terminal lever (*g*). The nut at the end (*d*) is adjusted to give the minimum load on the spring (*s*) by starting the spring (*s'*) when adjusted to a particular tension, and the cam

movement of (*d*) to its extreme downward position is made to give the requisite maximum stress in the specimen by adjusting the

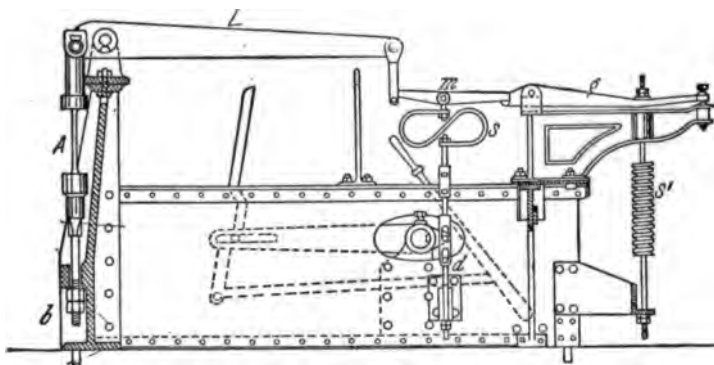


FIG. 24.—Wöhler's Machine for Repetitions of Tensile Stress.

spring (*s'*) so as to just lift at this position of (*d*). The rod is adjustable by means of a turnbuckle. In this way the bar (*A*) can be stressed

n between any chosen limits. Wöhler also designed special device for repeating bending stresses and for repeating torsional stresses.\*

**The White-Souther Endurance Testing Machine.**—In Fig. 25 shown a more recent device than those of Wöhler for imposing a repeated bending stress. The specimen (*BB*) is wedged firmly in the revolving pulley (*A*) by tapered liners, thus forming a cantilever on either side of the pulley. The loads are suspended on the rods (*EE*) which are supported by the bearings (*DD*) resting on the specimen. Speed counters (*CC*) are provided to indicate the number of revolutions made by each end of the specimen before rupture. The customary speed used on the machine is 1300 r.p.m. This apparatus takes a specially formed specimen about 14 in. long. The portion of the shaft within the pulley is  $\frac{1}{4}$  in. in diameter; the portion subjected to maximum stress has a diameter of  $\frac{1}{2}$  in.

**66. Koppers Repeated-stress Testing Machine.**—The repeated stress testing machine shown in Fig. 26 was designed by Prof. J. B. Koppers and built at the University of Wisconsin. The test consists of deflecting the specimen *A* on either side of the vertical position by means of the slider *C*, the number of cycles of stress for rupture being automatically recorded. It is possible to set the machine so that the specimen may be stressed within or beyond the elastic limit. When adjusted for the latter condition the machine works to best advantage.

The lever *B* is ordinarily not attached to the machine. It is for the purpose of measuring the load required to produce a given deflection. Slider *C* has mounted on it two auxiliary sliders *D*, which are accurately located by micrometers reading to 0.001 in. The sliders *D* carry hardened steel blocks which deflect the specimen. If these blocks are

\* See *Engineering*, Vol. 11, p. 199, 221, 224, etc.



FIG. 25.—A White-Souther Endurance Testing Machine.

moved farther apart than the diameter of the specimen, a slight amount of impact is given the test-piece. A scale mounted on the frame at *E* and a vernier mounted on the slider *C* permits the measurement of the stroke, and also the setting of the slider exactly at the middle of its stroke with an accuracy of 0.002 in. For most tests a true cylindrical specimen  $\frac{3}{8}$  in. in diameter and 9 in. long is used, a test being made on both ends. The length under test is 4 in.; although it is possible by inserting blocks at *K*, to obtain a longer length. The hardened steel grip-

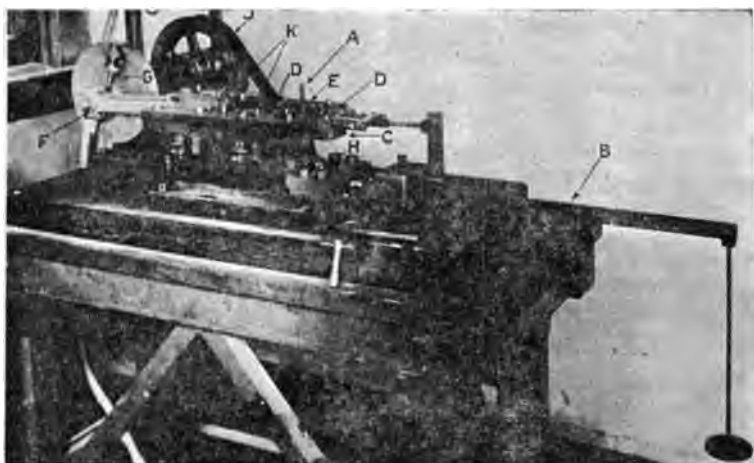


FIG. 26.—Kommer's Repeated-stress Testing Machine.

ping blocks *H* can be removed and others inserted so that forms other than rounds may be tested. The crank pin *F* may be roughly adjusted to give the desired stroke by means of a pointer and scale at *G*; more accurately by the scale *E*.

In testing, the machine is adjusted to give the desired deflection and impact. A specimen is then inserted and the flywheel *F* is started by a motor. When full speed is reached the test is begun by throwing a clutch at the end of the flywheel shaft. A lever which holds the end and of the specimen after rupture and also stops the counter automatically has been designed, but is not shown in the figure.

#### AUXILIARY APPLIANCES EMPLOYED IN LOADING SPECIMENS

**67. The Transmission of Load to a Specimen.**—In making tests of materials it is very desirable that the experimenter have control not only of the magnitude but also of the places of application and the directions of the forces applied to the specimen. Unless such conditions

obtain stresses of a different kind or of undesirable magnitude may be produced in the test-piece. In most tension and compression tests an axial load is sought; in most transverse tests a loading which produces bending in a given plane is desired. On account of the devices employed to transmit pressure from the testing machine to the specimen, a brief description of some of these appliances will not be given.

#### GRIPPING DEVICES FOR TENSION TESTS

**68. Wedges or Grips.**—By far the most frequently used device for holding tensile specimens in the jaws of the testing machine is a set of four serrated wedges. The flat wedges shown in Fig. 27 are for speci-

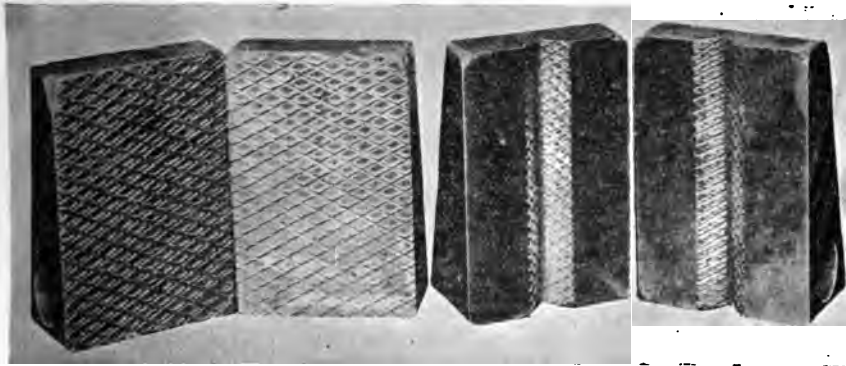


Fig. 27.—Flat and Grooved Wedges for Tensile Tests.

mens of rectangular cross-section; the grooved wedges are used with cylindrical test-pieces.

Liners are placed in back of the grips in testing thin specimens. Fig. 28*d* shows the proper method of gripping a test-piece when wedges are employed. To prevent sticking in the jaws of the machine the backs of the grips should be coated with a heavy lubricant.

Wedges are often objected to on the ground that they crush the specimen and pull unevenly on opposite sides of a test-piece, thus producing bending or oblique stresses. However, for rough commercial testing experience has proven such grips to be satisfactory.

**69. Spherical Seated Holders.**—In scientific testing to determine the elastic limit of a material, it is desirable to secure as close an approach to an axial loading as possible, and a method more refined than indicated above should be employed. For such tests spherical seated holders of the type shown in Fig. 29 have been much used.



The efficiency of this device is dependent both upon the skill of the mechanic in making the appliance and in fashioning the specimen, and upon the care exercised in using the device. To reduce the frictional resistance of the spherical seat to a minimum, the radius of the sphere should be made as small as a proper consideration of the crushing strength of the ball will permit, the ball and seat should be ground to fit, and the bearing should be lubricated with a thin film of oil. Even with a well-made holder and careful manipulation, extensometer measurements taken on opposite sides of a specimen will often differ considerably. This dif-

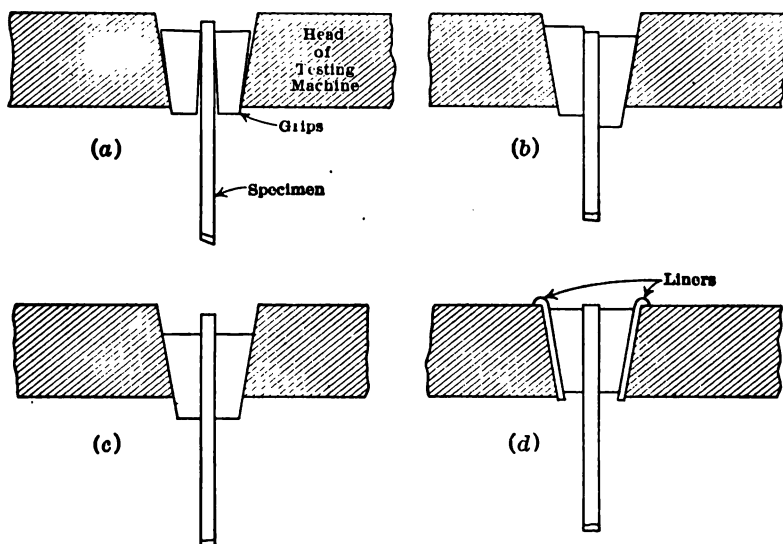


FIG. 28.—Correct (d) and Incorrect (a, b, c) Methods of Gripping Tensile Specimens with Wedges.

ficulty may be partly removed by applying a small load, reading the extensions, then removing the load and adjusting the specimen slightly to overcome the eccentricity.

A type of holder which is suitable for use with tempered steel specimens is shown in Fig. 30. Portion *A* is a socket nut which is threaded to fit the lower end of the bolt *B*. The latter may be provided with a spherical head or threaded at the upper end to fit the holder of Fig. 29. After the head of the test-piece has been inserted through the hole in portion *A*, the halves of the spherical-seated washer *C* are slipped about the specimens and held in place by a rubber band or a split ring. Parts *A* and *B* are screwed together, and then the lower end of the specimen is socketed.

**C**      **I**      : Suspension.—Fig. 31 shows a form of crossed  
 4      :      n which has been successfully used at the Materials

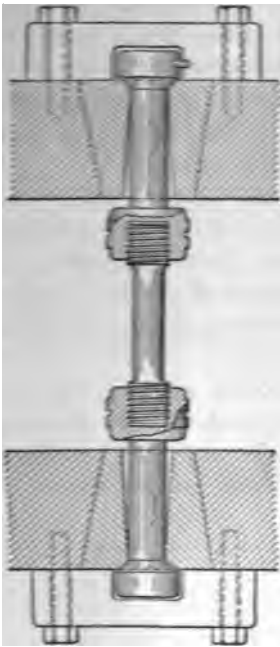


FIG. 29.—Holders with Spherical Seats for Tensile Tests.

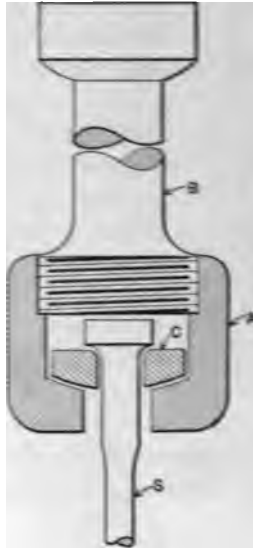


FIG. 30.—Spherical-seated Sockets for Hard Steel Specimens. (After K. W. Zimmerscheid in *Trans. Soc. Auto. Engr.*, Vol. 8, pt. 2, p. 162.)

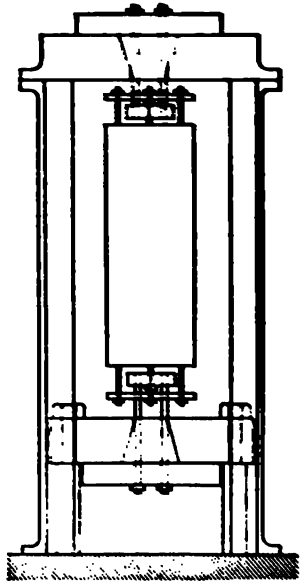


FIG. 31.—A Crossed Knife-edge Suspension for Testing Concrete Cylinder in Tension.

Testing Laboratory at the University of Wisconsin in making tension tests of concrete specimens.

## LOADING APPLIANCES FOR COMPRESSION TESTS

**71. Rigid Bearing Blocks.**—In rough testing compressive loads are applied through heavy blocks one of which is fastened to the lower side of the pulling head of the testing machine and the other is supported by the weighing table. With such bearing blocks it is very difficult to secure an axial load on the specimen.

**72. Adjustable Bearing Blocks.**—For carefully conducted tests various appliances have been devised for imposing compressive loads. The spherical bearing block, shown in Fig. 32, has been extensively used. Some experimenters prefer to support this device on the weighing table beneath the specimen, others hang it from the moving head above the specimen.

Ball-bearing blocks have also been widely used in compression tests. Fig. 33 shows a form of block which is satisfactory for such tests.

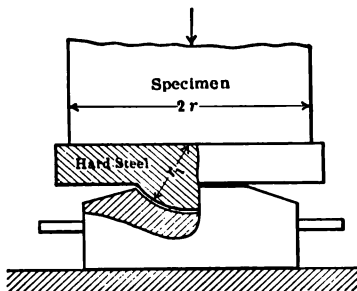


FIG. 32.—A Spherical Bearing Block.  
(Ordinarily  $r \div r_1$  varies between 1 and 2.)

In using any of the above devices great care must be exercised to make the axes of the bearing block, specimen, and testing machine collinear. If deformation measurements are being taken, the load on the specimen may be made axial in the following manner:

Readings of the shortening of elements on opposite sides of the specimen may be taken for moderate loads and the specimen adjusted on the bearing block until the deformations of all elements are equal.

In testing columns for the purpose of checking existing theories, most investigators have attempted to make the ends of the specimen free to revolve. Professors Bauschinger, Tetmajer and Lilly mounted their columns on cone or knife-edge bearings at the gravity axes. M.

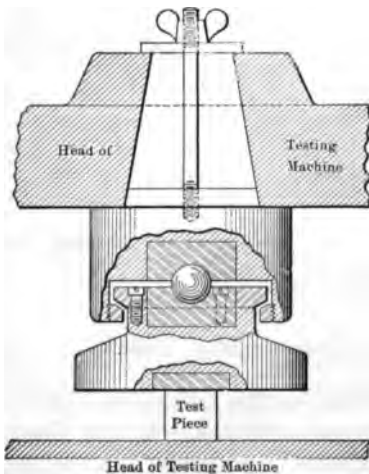


FIG. 33.—Ball-bearing Block for Compression Tests.

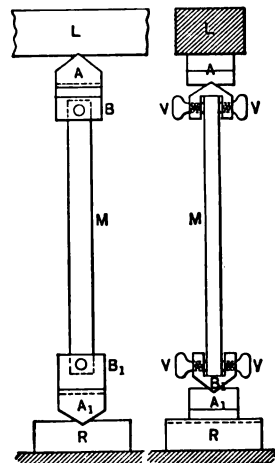


FIG. 34.—Considère's Mounting for Column Tests.

Considère \* mounted his specimens on crossed knife-edge bearings provided with lateral-screw adjustments, as shown in Fig. 34, and arranged a very delicate electric contact which would indicate a lateral deflection

\* Ingénieur-en-chef des Ponts et Chaussées, France; see *Report of the French Commission on Methods of Testing Engineering Materials*, 1895, Vol. 3, p. 124.

of 0.001 mm. He then applied moderate loads and adjusted the bearings until the columns remained absolutely vertical. With such a device the ends of a column can be made practically free; and furthermore, the load may be applied at the true axis of the test-piece.

### SUPPORTING AND LOADING DEVICES FOR TRANSVERSE TESTS

**73. V-blocks.**—Commonly the supporting tools for transverse tests are inverted V-blocks which are either fixed to the bed of the testing

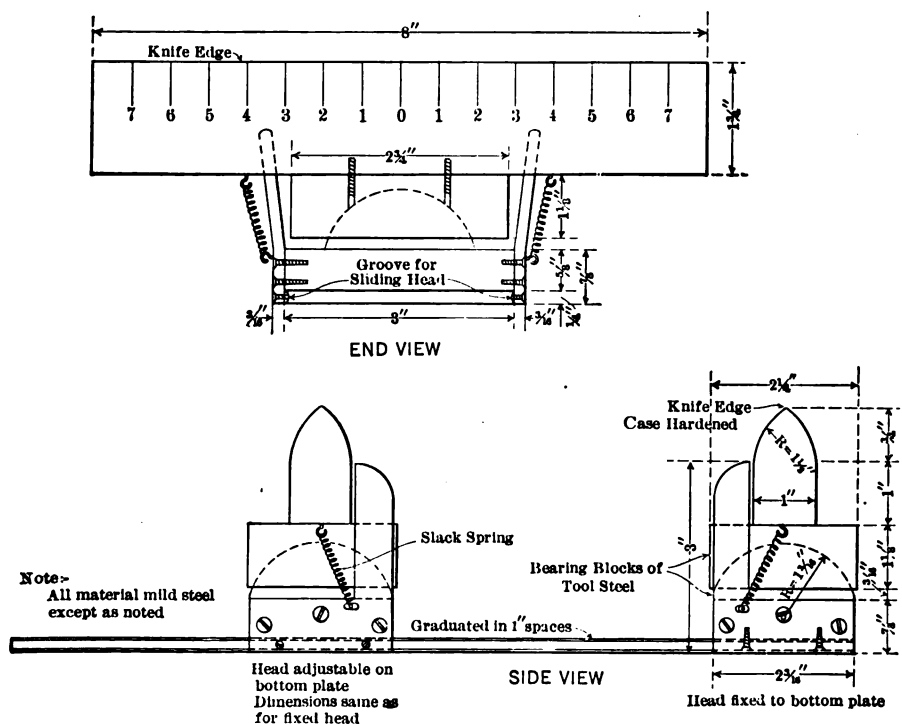


FIG. 35.—Knife Edges with Spherical Bearings for Cross-bending Tests on Brick.

machine or which may be slid into the desired position. The edges of the blocks are rounded to produce line contact across the beam, and are surmounted by metal bearing plates when tests of non-metallic specimens are to be made. Through a similar V-block fixed in the jaws of the movable head of the testing machine and bearing upon a metal plate or saddle, the load is imposed upon the test-piece. Obviously, with the above arrangement the beam is subjected to more or less longitudinal restraint. Furthermore, if the lines of contact on the beam are not parallel to the edges of the V-blocks, the test-piece will be subjected to torsion.

**74. Adjustable Bearing Blocks.**—To avoid the errors encountered in the use of fixed V-blocks it has been customary to mount the beam on adjustable supports. A much-used form of support consists of a V-block



Fig. 36.—An Adjustable Supporting Block Used in Testing Large Beams.

mounted on a spherical seat. Fig. 35 shows such a device which has been employed considerably at the University of Wisconsin in testing small specimens. Fig. 36 illustrates the form of support which Tinius Olsen & Co. advocate for testing large beams.

In careful work it is advisable to transmit the load from the pulling head to the specimen, or to the auxiliary beams used in loading, through a spherical bearing block. If the specimen is loaded at more than one point, rollers resting on hard steel plates

should be employed to apply the pressure of the auxiliary beams upon the test-piece.

## BEDMENTS

**75. The Use of Bedments.**—Even though adjustable bearings be employed, it is necessary that the contact surfaces of the test-piece be plane and approximately parallel. Metal specimens should always be machined or ground until the above conditions are fulfilled. For the best results, specimens of stone, brick, or concrete should also be ground. However, on account of the length of time and the equipment required to grind specimens, many experimenters bed such test-pieces in some material which will lessen the effects of surface inequalities. If a bedment is used it should be made as thin as possible; also, it should be composed of a substance which will not spread or flow under pressure and which possesses the same strength and elastic properties as the specimen. The latter condition is difficult to satisfy and is, unfortunately, often disregarded.

**76. Plaster of Paris.**—Bedments of plaster of Paris have long been used. In making such bedments it is good practice to place a sheet of sized paper between the plaster of Paris paste and the specimen to prevent the absorption of water by the latter, since this action invariably affects the strength of the test-piece. A small load should be immediately imposed upon the specimen while the bedment is soft and allowed

to remain until the plaster of Paris has set before proceeding with the test.

**77. Cement Mortar.**—At some laboratories it has been the practice to bed concrete and mortar compression specimens in neat cement paste or in a rich mortar made with fine sand. Such bedments must be made long enough before the time of testing to permit the paste or mortar to properly harden. In general, only the tops of the specimens need be treated, the procedure being as follows: The specimen is placed upon a leveled surface and a thin coat of plastic mortar applied to the top. The bedment is then finished by pressing down upon it a piece of plate glass which is carefully trued with a spirit level. By adding 5 to 10 per cent. of plaster of Paris to neat Portland cement the time of setting for the bedment may be greatly lessened.

**78. Miscellaneous Bedments.**—Blotting paper, bristol board, and cardboard have been considerably used for bedments. The virtue of these is open to question. Lead or rubber should never be used as these substances spread under pressure and decrease the resistance of the specimen.

## APPLIANCES FOR MEASURING DEFORMATIONS

### EXTENSOMETERS

**79. Essential Features of Extensometers.**—For measuring the elastic extension of materials subjected to tensile loads, a great variety of devices have been employed. Four principal types of extensometers are micrometer-screw with electric-contact, indicating dial, multiplying lever, and mirror apparatus. In any type of extensometer the following conditions should obtain:

1. The apparatus should be directly attached to the test-piece.
2. It should be arranged to measure deformations on opposite sides of the specimen if the change in length of the axis is to be determined. If the maximum deformation of any element is desired, measurements along at least three elements, preferably 120 deg. apart, are required.
3. The portions of the apparatus transmitting deformations should be parallel to and equidistant from the axis of the specimen. If only two measurements are made these should be along lines in the plane of the axis and equidistant from it.
4. The apparatus should be so arranged that the relative positions of all parts with respect to the axis of the specimen will remain the same throughout the test.
5. For precise measurements the instrument should read to at least 0.0001 in. and should be sensitive to half that amount. To de mine

the accuracy of the instrument, its readings should be checked by comparison with a standardized measuring device.

6. The apparatus should be so constructed that it may be quickly applied to or removed from a specimen without interfering with the application of the load.

**80. A Micrometer-screw Electric-contact Extensometer.**—In Fig. 37 is shown a simple form of micrometer-screw extensometer reading to 0.0001 in. The upper and lower portions of the apparatus form parts

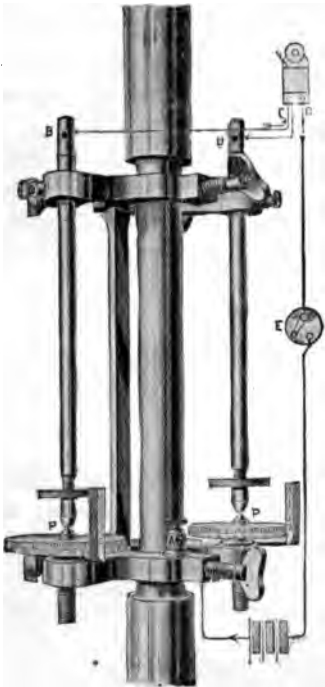


FIG. 37.—A Micrometer-screw Electric-contact Extensometer.

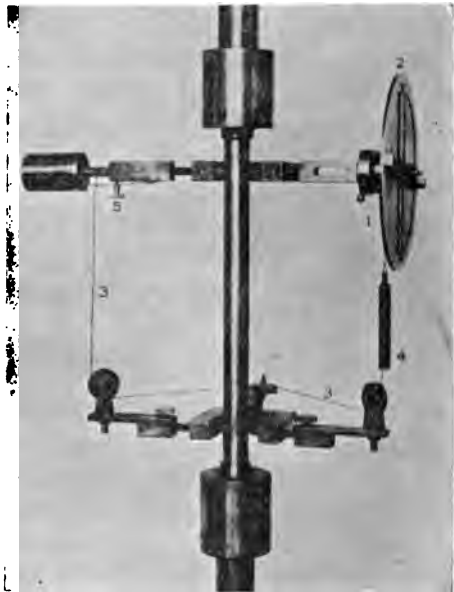


FIG. 38.—A Wire-wound Dial Extensometer.

of an open electric circuit. Contact between a micrometer-screw and rod is announced by the bell which rings when the circuit is closed. Only a small constant current is used, since a large current will jump an appreciable gap between screw and rod. On account of the uncertainty of bell mechanisms telephone receivers are often substituted. To prevent a closed circuit through the specimen the vertical rods are insulated by gutta percha sleeves placed in the top collar of the instrument.

This common type of extensometer is quite satisfactory for student work, and with skillful handling will give accurate results. In operating the screws on this instrument, however, great care must always be exer-

cised to make the plane of the turning couple exerted by the fingers perpendicular to the axis of the screw.

**81. A Wire-wound Dial Extensometer.**—A measuring device which has a wide range of application is the indicating dial. Fig. 38 shows an inexpensive form of dial and the method of mounting employed at the University of Wisconsin. The dial itself (1) consists of a brass disc 5 in. in diameter with a nickel-plated rim graduated into 1000 parts. A pointer (2) with a vernier on one end is mounted on a spindle provided with well-made adjustable pivot-bearings. One of these is located in the cross-bar in front of the disc, the other is placed in the bracket on the rear. The spindle also carries on the back side of the disc a drum 1 in. in circumference. Twice the relative displacement of the lower collar of the extensometer with respect to the upper is transmitted through a No. 38 covered coppered wire (3) to the drum on the dial. The small weight (4) serves to keep the wire taut. The pointer may be set to read zero by means of the adjusting screw and spring (5). By means of the vernier, deformations of the specimen can be read to 0.0001 in.

Some experimenters use two dials and replace the wires by sliding rods which are pressed against the dial drums. The rods are more readily adjusted, but on the other hand they slip more easily than the wires.

In setting up this apparatus, care must always be taken to place the collars in planes perpendicular to the specimen, so that the motion of the collar arms caused by lateral contraction of the specimen will have no component along the axis of the test-piece. The chief advantages of this type of extensometer are its adaptability and the fact that it is self-indicating for large or small deformations.

**82. Multiplying Lever Extensometers.**—A simple and accurate self-indicating extensometer is Marten's modification of the Kennedy extensometer, sketched in Fig. 39. The apparatus consists of two L-shaped bars (1) which are held against opposite sides of the specimen by a spring (2). The rhombic fulcrum (3), pivoted in shallow seats formed in the

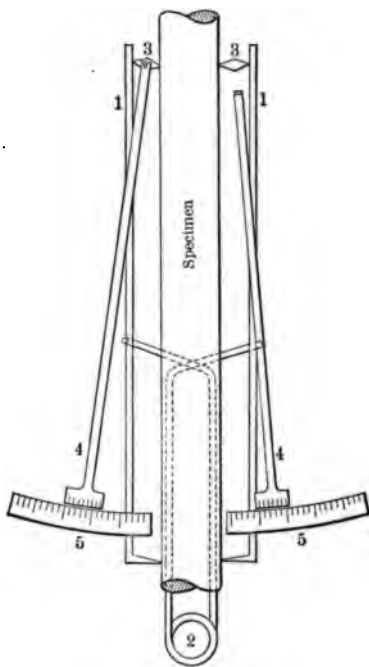


FIG. 39.—Sketch of Kennedy-Martens Multiplying Lever Extensometer.



bars (1), carry the levers (4). The ends of the levers move along graduated arcs (5) attached to the bars (1). With a 50 to 1 multiplication, readings may readily be made with this apparatus to 0.0001 in.

An apparatus embodying both a multiplying lever and an indicating dial is the Berry \* strain gage shown in Fig. 40. This device is particularly well adapted to the measurement of deformations in portions of a structure. Any number of measurements on different portions of a beam, column or floor may be made with one instrument provided a pair of  $\frac{1}{16}$  in. holes, spaced the gage length of the apparatus apart, are

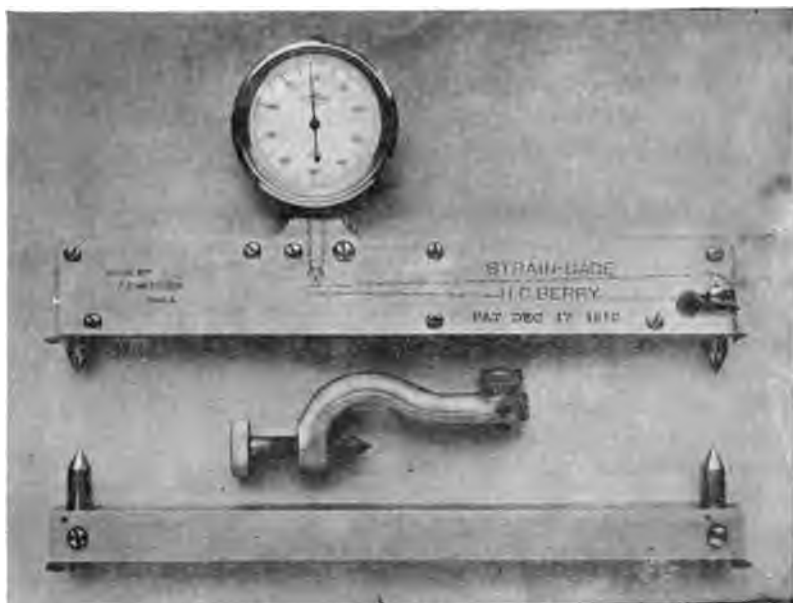


FIG. 40.—Berry Strain-gage for Measurements over 8-inch Lengths, Clamp for Attaching Apparatus to Specimen and Center Punch.

drilled a short distance into each element at the proper place. In measuring, the left pivot which is rigidly attached to the invar-steel side bars is placed in one of the holes and the right pivot, which terminates the short arm of a five-to-one bell-crank lever, is adjusted to the other hole. The long arm of the bell crank rests against the pin of an Ames dial reading to 0.001 in. The instrument is placed in the pair of holes several times and the average reading of the dial noted; the load is changed and the reading repeated. By exercising great care readings accurate to 0.0001 in. may be taken with this device. Side bars for 2, 8 and 20 in. gage lengths can be procured with this instrument.

\* Described in *Eng. Record*, June 11, 1910.

**83. Martens' Mirror Extensometer.**—For greater refinement than 0.00005 in. in the measurement of deformations some form of optical lever is generally employed. One of the most accurate and adaptable instruments is the mirror apparatus devised by Martens. The sketch of this apparatus, Fig. 41, indicates the principle of operation. In this instrument the multiplying levers shown in Fig. 39 are replaced by small mirrors, (4) (Fig. 41), which are attached to the rhombic fulcrum (3) so that the axis of each fulcrum passes through the reflecting plane of its mirror. The deformation of the specimen causes slight rotations of the mirrors. The latter are determined by observing successive positions of the cross-hairs in the telescopes (5) with respect to the images of the scales (6), thus finding  $e_r$  and  $e_f$ .

A better idea of the arrangement of the fulcrum and mirrors can be

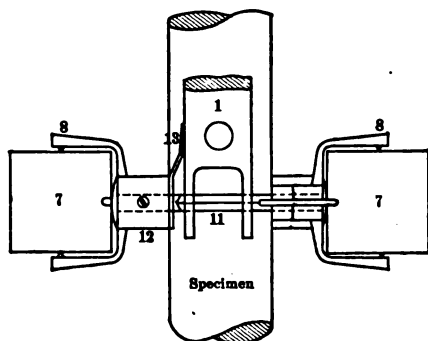


FIG. 41.—Diagrammatic Sketch of Martens' Mirror Extensometer.

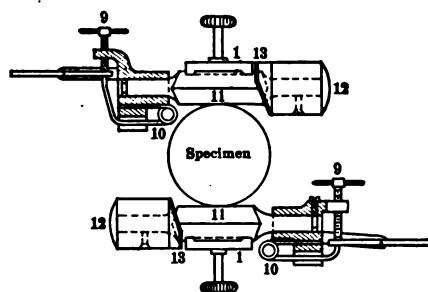


FIG. 42.—Fulcrum and Mirrors of Martens' Extensometer.

gotten from Fig. 42, which presents a view seen from the telescope. The mirrors (7) may be adjusted about a vertical line through the pivots of the frames (8) by means of screws (9). The springs (10) hold the mirrors against the screws. The mirrors may also be turned about the spindles connecting them with the fulcrum (11). To balance the weight of the mirror each fulcrum is provided with a counterweight (12). By making the vertical arms (13) parallel to the bars (1), the positions of the fulcrum (11) may be accurately adjusted to the gage length for which bars were designed.

The entire apparatus is very light and may be quickly placed upon

the test-piece. If a flat scale is employed, proper correction to the readings must, of course, be made. With careful handling readings to 0.000002 in. can be taken with this instrument.\*

**84. Autographic Stress-diagram Appliances.**—These fall into two general classes:

1. Those in which the load coordinate is recorded through a movement of the poise on the scalebeam.

2. Those in which the load coordinate is recorded through the lifting of the scalebeam against the increasing resistance of a calibrated spring attached to its free end.

The deformation coordinate is in all cases multiplied either by lever or by the principle of the cone pulley. The paper is usually attached to a cylinder, although it has sometimes been attached to a plane board. Generally the pencil moves in a straight line, indicating one of the two coordinates, while the cylinder (or board) moves to register the other function, and it matters not which of the two movements is made by the deformation of the specimen and which by the increasing load. The location of the paper and its mountings is a matter of convenience simply. If cords (or wires) are used to transmit the stretch of the specimen, they must form a pair, symmetrically placed on opposite sides of the specimen; they must be attached to one collar and pass through pulleys similarly placed on the other. They should then pass off in a plane at right angles to the specimen † and connect with the ends of an "evener" (lever), to the center of which is attached the single cord which passes either to the pencil-holder or to the cylinder which carries the paper. If cords are used, they should be such as do not stretch appreciably for such changes of stress as occur in them during the test.

Fig. 43 shows the Riehlé automatic and autographic recording attachments placed on a 200,000-lb. machine. The deformation of the specimen is measured as follows: U-clamps attached to the test piece at the proper gage marks by means of sharp pointed thumb screws support the fingers on the outstanding ends of the horizontal arms. Both arms are supported by a set of telescoping tubes sliding on the vertical shaft. The upper and lower set screws on these tubes are loose but the middle one is fast during a test. With this arrangement any elongation of the specimen produces an equal displacement of the lower arm with respect to the upper, but slipping of the specimen in the grips has no effect. The elongation is transmitted through the vertical rack and pinion on the lower arm to

\* For a more complete description and discussion of the Martens mirror apparatus see *Martens' Handbook of Testing Materials*, translated by G. C. Henning. The apparatus is made by J. Amsler-Laffon & Sons, Schaffhausen, Switzerland.

† This is necessary in order that the stretch of the specimen may be fully represented in the shortening up of the cord. The cords should therefore be attached to the moving end of the specimen.

of miter gears which cause a point on the surface of drum *B* to turn through a distance five times as great as the stretch of the specimen. On the opposite side of the drum is a vertical screw which is geared to the poise beam and drives a nut carrying the recording pencil. Lost motion in the rotating parts is taken up by attaching a weighted cord at the lower end of the drum as shown.

Automatic control of the poise is effected through the device at *A*. A round belt to the left of *A* is connected to the driving pulley of the machine and turns the small horizontal shaft just below *A*. At the right

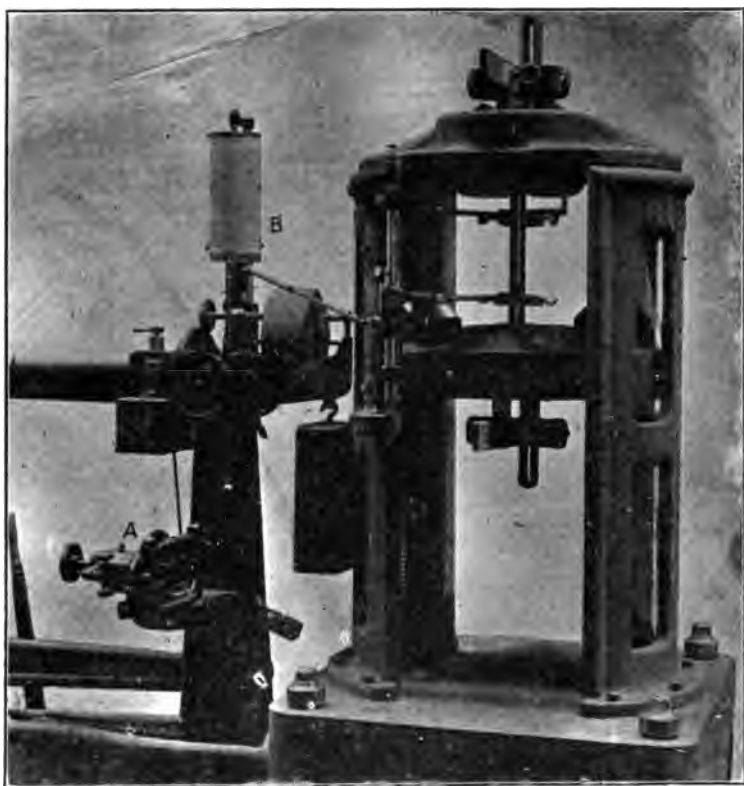


FIG. 43.—Riché Automatic and Autographic Recording Attachment.

of this shaft is attached a cast iron disc which drives either of the small fiber wheels placed equidistant from its center. The speed of the fiber wheels is regulated by changing their positions with respect to the center of the disc. A pair of magnets, one of which is shown in the figure, is provided for pulling each fiber wheel against the cast iron disc, and the frame carrying the fiber wheels is connected by means of sheaves and a belt to the poise screw. When the scalebeam rises it makes an

electric contact and one of the fiber wheels is forced against the disc, thus causing the poise to move outward and the pencil upward. When the beam drops and hits the lower contact, the poise and pencil are moved in the opposite direction.

From Fig. 44 it is evident that the stress-diagrams gotten from this apparatus show yield point, partial elongation, maximum load, and general

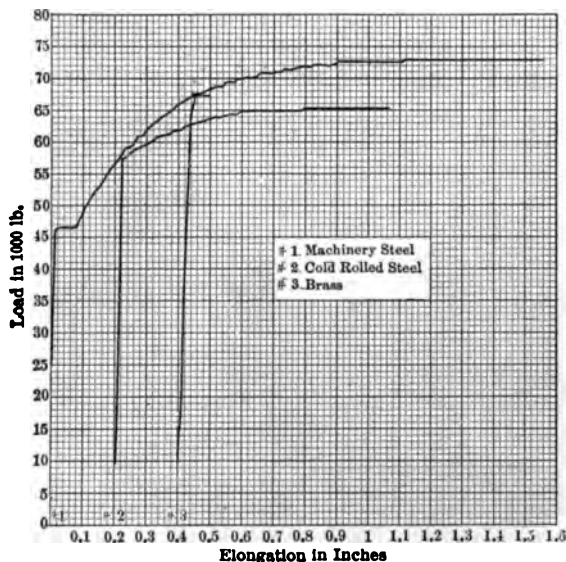


FIG. 44.—Autographic Stress-diagrams Made by Riehle Bros. Test. Mach. Co., with Device Shown in Fig. 43.

Specimens were approximately 1 in. in diameter; gage length was 8 in. Diagram is about one-third true size.

shape of curve very well. They are not, however, sufficiently precise for finding the modulus of elasticity or limit of proportionality.

#### COMPRESSOMETERS

**85. Essential Features of Compressometers.**—The conditions mentioned under essential features of extensometers apply with equal force to compressometers; and the use of an apparatus which measures the relative displacement of the bearing surfaces on either end of the specimen should never be permitted if the modulus of elasticity of the test-piece is sought.

**86. Brief Discussion of Various Types of Compressometers.**—Inasmuch as the principles of measurement are the same for both extensometers and compressometers, only types of the latter in which the method of attachment differs from that employed in the previously illustrated extensometers will be considered.

Fig. 45 represents Olsen's cylindrical extension and compression

electric-contact micrometer. A similar apparatus is built for rectangular specimens. The upper and lower collars of either device are provided with four points of contact and the apparatus for specimens of any size or length. As shown, the gage frame is in place upon the specimen; this must be removed before a compression test is begun. The apparatus reads to 0.0001 in.

In Fig. 46 is shown a wire-wound dial compressometer used at the University of Wisconsin in testing concrete cylinders and columns. The



FIG. 45.—A Micrometer Screw Electric-contact Compressometer.

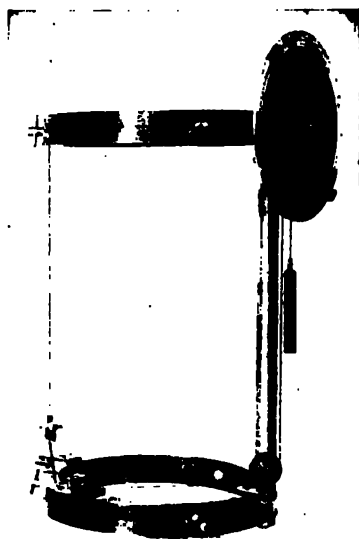


FIG. 46.—Wire-wound Dial Compressometer for Cylinders and Columns.

split rings at the top and bottom have three-point contact and dials may be mounted on these to measure deformations along two or more lines parallel to the axis of the test-piece. This apparatus also measures to 0.0001 in.

## DEFLECTOMETERS

**87. Essential Features of Deflectometers.**—To measure bending of beams, columns, floors and other elements of structures an instrument called a deflectometer is employed. The essential conditions which should obtain in a deflectometer designed for accurate measurements are:

1. The apparatus should indicate the relative deflections of points in the neutral surface of the member. In many forms of deflectometers

the apparatus itself is suspended from the neutral surface at four points directly above the supports and the deflections measured with respect to a plane through the points.

2. The parts of the deflectometer forming the datum to which the deflections are referred should be unstressed. This principle is often violated. A common practice is to measure deflections with reference to the bed of the testing machine, assuming this to be rigid (see Fig. 48).

3. Provision should be made for determining the deflections of both sides of the test-piece.

4. For most work an apparatus which is sensitive and accurate to 0.001 in. will be found satisfactory.

**88. A Dial Deflectometer.**—Fig. 47 represents a dial deflectometer which has proven to be a very satisfactory instrument for measuring small deflections. The side bars (1) forming the datum plane of the device are freely supported on pins driven into the neutral surface above

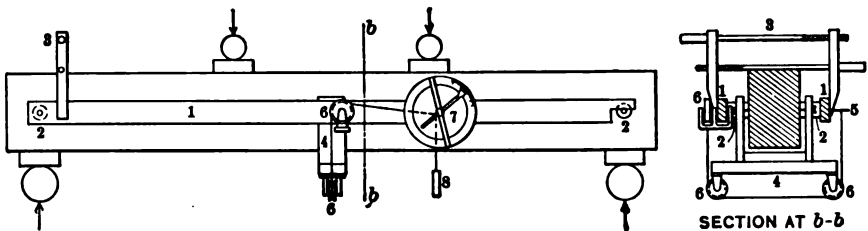


FIG. 47.—Wire-wound Dial Deflectometer Used at the University of Wisconsin.

the end bearings. To prevent the bars from rubbing against the sides of the specimens washers (2) are inserted as indicated. Clamp (3) holds the side bars in place and prevents them from vibrating during the test. Yoke (4) is clamped by means of thumb-screws to the neutral surface at the point whose deflection is to be measured. In transmitting the motion of the yoke to the dial on the side bars, use is made of No. 38 covered copper wire. One end of the wire is attached to the pin (5) driven into the side bar shown at the right of the sectional view. The wire is then carried around the three idler pulleys (6) and wrapped around the drum of dial (7). Weight (8) serves to hold the wire taut. If the parts of the apparatus are properly arranged the increments in dial readings will be twice the deflection of the beam. If the deflection of each side is desired the device can be readily modified to meet such requirement. With this instrument readings of deflections to 0.0001 in. may be made.

**89. Multiplying-lever Deflectometer.**—Fig. 48 illustrates a common type of lever deflectometer reading to 0.001 inch. Frequently, this device is so arranged that the movement of a point on the lower side

of the beam with respect to a point in the base of the testing machine is gotten. Unless the deflection of the base of the machine is known to be of no consequence, allowance should be made for it. Furthermore, the indentation of the specimen at the supports enters into measurements made in this manner.

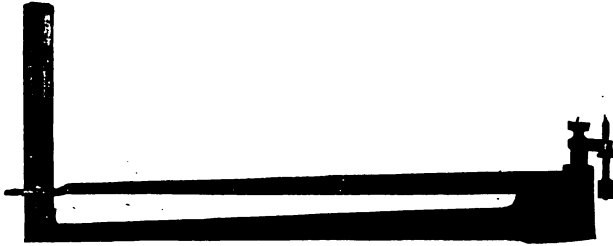


FIG. 48.—A Multiplying Lever Deflectometer with Adjustable Short Arm.

Other convenient devices employing the lever principle are illustrated in Fig. 9 and Fig. 10.

**90. A Wire-mirror-scale Deflectometer.**—A very simple and convenient form of deflectometer for tests in which large deflections are to be measured is indicated in Fig. 49. In careful work the opposite side of the beam should be equipped with a duplicate of the appliance shown.

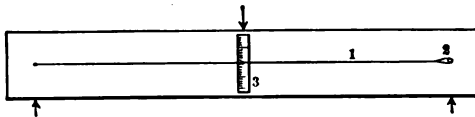


FIG. 49.—A Wire-mirror-scale Deflectometer.

The fine wire (1) which is kept taut by the rubber band (2) is attached to pins placed in the neutral surface above the supports and forms the datum from which deflections are measured. The highly polished scale (3), ordinarily graduated to 0.01 in., is also suspended from the neutral surface. In reading, the observer brings his eye into the plane of the wire and its image and notes the division intercepted on the scale.

## BEAM DEFORMETERS

**91. Wire-wound Dial Deformeters.**—The device illustrated in Fig. 50, which is a modification of the extensometer in Fig. 36, is employed to measure the deformations in the longitudinal fibers of a beam. Generally such measurements are made upon the top and bottom fibers. If fibers nearer the neutral surface are to be measured, deeper U-shaped clamps must be provided.



**92. Other Types of Deformers.**—Many experimenters have employed apparatus of the type shown in Fig. 51. Some have replaced the micrometer-screw by dials with friction rollers. An objection to

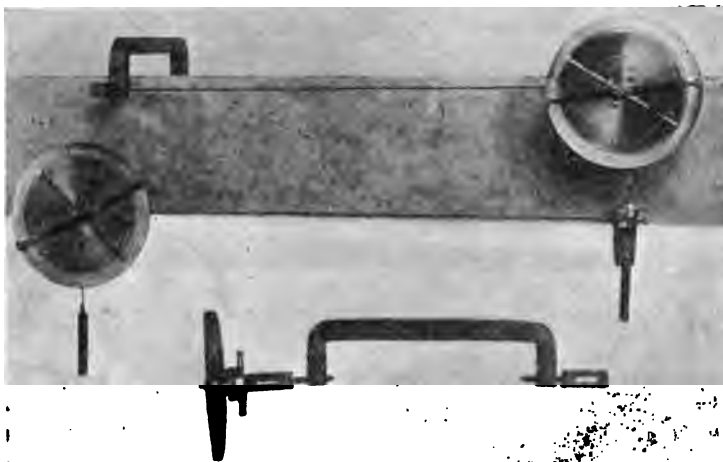


FIG. 50.—Wire-wound Dial Deformer Device Used to Measure Fiber Stresses in Beams.

the form of frame shown is that free motion of the upper and lower points of contact is restricted more or less depending on the rigidity of the frames.



FIG. 51.—Micrometer-screw Deformers.

The Berry strain-gage shown in Fig. 40 is a very useful device for measurements of all sorts of deformations in beams.

## DETRUSION INDICATORS

**93. Porter's Detrusion Indicator.\***—Essentially, this apparatus consists of two rings, each of which is clamped to the specimen by three

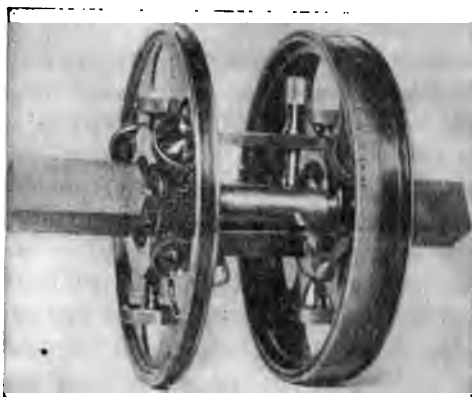


FIG. 52.—Porter's Indicator with Distance Bars in Place.

set-screws, see Fig. 52. The distance between the rings is fixed by gage bars provided with studs for centering the test-piece. The right-hand ring is graduated in degrees and supports, on a ball-bearing, a concentric

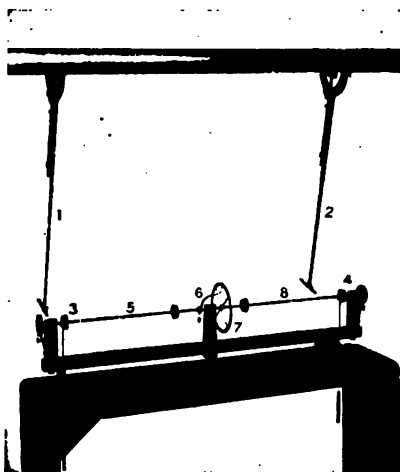


FIG. 53.—Torsion Indicator Used at the University of Wisconsin.

ring provided with a vernier reading to 5 minutes. The vernier ring is moved by a finger attached to the left-hand ring. In order that the

\* Described in *Proc. A. S. T. M.*, Vol. 10, p. 578.

parts of the apparatus may be free to move as the specimen suffers change in length, a ball joint is provided to connect finger and vernier ring. For setting the vernier to read zero the connection between the finger and vernier ring is equipped with a slow-motion tangent screw. The instrument shown was made for specimens less than  $1\frac{1}{4}$  in. in diameter and a gage length of 3 in.

**94. A Dial Indicator of Detrusion.**—An apparatus for detrusion measurements which has been in use for several years at the University of Wisconsin is illustrated in Fig. 53. The twist of the section of the specimen between the arms (1) and (2) is transmitted through No. 38 covered copper wire to the drums (3) and (4), respectively. The spindle (5) carrying drum (3) actuates the pointer (6). Similarly dial (7) is connected to drum (4) by spindle (8). From this arrangement it is, therefore, evident that the twist between the two sections of the specimen can be gotten by noting the relative positions of pointer and dial corresponding to the increment in torque. In the apparatus shown the multiplying factor is 40, and the dial is graduated in one-half degrees. Any deflection produced by bending of the specimen during the test will, of course, affect the readings of this apparatus. However, a rough computation quickly demonstrates that such effects are negligible.

#### MISCELLANEOUS APPARATUSES FOR MEASURING DEFORMATIONS

**95. Multiplying Dividers.**—A very efficient little device for accurately locating the yield point in a tension or compression test is Capp's



FIG. 54.—Capp's Multiplying Dividers for Detecting the Yield Point.

multiplying dividers, shown in Fig. 54. In performing a test with this instrument, the operator grasps the cupped pivot-heads between the thumb and finger of his left hand and places the hard steel points on the end of the short arms in punch marks spaced 2 in. apart. By this method the operator's right hand is free to move the poise on the scale-beam if the machine is not provided with an automatic drive. For elastic stresses the motion of the pointer over the scale is hardly appreciable, but when the yield point is reached the rapid increase in the rate of motion of the pointer instantly warns the operator of the fact.

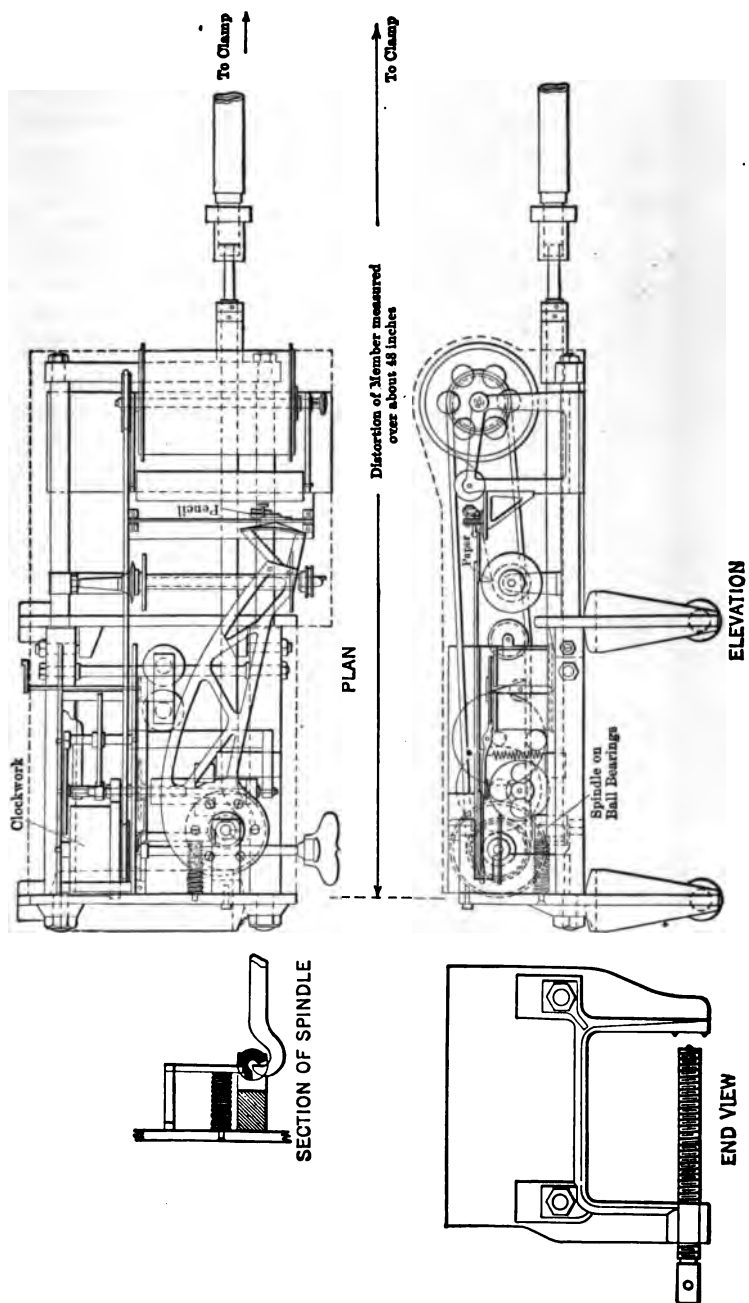


FIG. 55.—A Recording Extensometer for Measuring Deformations in Bridge Members.

With such a device mistakes in the location of the yield point due to slipping of the grips are obviated. In tests of high-carbon steels, wire, and other materials having a high yield point the drop-of-beam method is very uncertain at best. By using the divider method, however, a distinct indication of the yield point can be readily obtained.



(b)

FIG. 56.—A Wire Rope Extensometer. (a) View from Telescope; (b) Clamp for Attaching Scales to Rope.

**96. A Recording Bridge Deformeter.**—The apparatus shown in Fig. 55 was designed by the Structural Engineering Department of the University of Wisconsin to measure the deformations in bridge members while subjected to moving loads. The movement between the gage points of the apparatus is transmitted through a long rod to a light, rigid lever actuating a pencil point which records 50 times the deformation on the diagram sheet. The latter is wound about the drum which is turned by an electrically controlled clockwork. A number of these instruments were used with success in a long series of experiments by the American Railway Engineering and Maintenance of Way Association.\*

**97. A Wire-Rope Extensometer.**—A simple and durable device for measuring the elongation of wire-rope is illustrated in Fig. 56. Increments of elongation are determined by taking simultaneous readings on the upper and lower pairs of scales by transits or telescopes set up a short distance from the test-piece. The scales are graduated to 0.01 in., and readings to half-hundredths may be established. Twisting of the rope does not materially affect the results if each telescope be placed on a level with the corresponding pair of scales. Furthermore, since the device is not delicate or expensive and the observer well back from the test-piece, readings may be taken until the specimen fails.

\* *Bulletin of American Railway Engineering and Maintenance of Way Association* No. 125, p. 9; also *Engineering News*, June 20, 1907. This apparatus is made by A. Wissler Instrument Works, St. Louis, Mo.

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## CHAPTER III

### THE MECHANICAL TESTING OF STRUCTURAL MATERIALS

**98. General Observations.**—Mechanical tests are those used to discover the qualities of the materials of construction under the action of external forces. Such tests, if they are to be of most value, should be made under conditions approximating as closely as possible those of practice. By standardizing these conditions the results become comparable wherever or by whomsoever they are made and are of very great importance in determining the properties and value of building materials. If such results can be made wholly independent of the means employed in making the tests, and hence to furnish a knowledge of the true characteristics of the material, they can be used safely in theoretical generalizations on the one hand, and in the practical designing of structures on the other. With many kinds of tests this ideal divorcement of the results from the conditions of the tests can certainly never be attained, as in the case of tests by impact, but it doubtless can be practically attained in some of the more simple tests, as in tension and compression. In the former case the most that can be accomplished is to prescribe uniform conditions in order that the results obtained by different experimenters may be comparable, although they may not serve for accurate scientific generalizations. They might also serve to give a relative value to the various materials or samples so tested, and to grade them with some degree of approximation to their true relative merits for a proposed purpose. Such tests, therefore, may serve fully their immediate object even though the results can be given no absolute significance whatever. If, however, the conditions of such tests are allowed to vary, they lose even this relative significance, and therefore become quite worthless. The standardizing of any particular kind of test evidently depends on the state of the science at the time; and as our knowledge of any particular property of a material increases, it is probable that our standard methods of testing will also have to change. No such standards, therefore, can be fixed permanently, but certain methods can be agreed on and followed for a time, and when a change is made let all change together. To attain to this kind of unity of action it is necessary to have a world's representative body which will command the confidence and allegiance of both the theoretical and the practical users of materials in all civilized countries to decide such questions.



The efforts of the International Association for Testing Materials towards securing such unity of action in various countries has not thus far been productive of great reforms although, the outlook is hopeful. However, some of the affiliated societies like our American Society for Testing Materials have been instrumental in unifying the methods of testing and standardizing specifications in force in their respective countries.

Evidently no complete standardization can be effected for tests on entire structural forms, since these vary in shape, size, and disposition of parts, but specimen tests can be standardized since all significant conditions can be made uniform.

**99. Mechanical Tests Classified.**—In a general way we may divide mechanical tests of building materials into the following classes:

With reference to the method of applying the loads we have—

(1) *Static Tests*, or those made with gradually increasing loads, such as the ordinary tests in tension, compression, cross-bending, torsion, and shearing.

(2) *Dynamic Tests*, or those made with suddenly applied loads, as by a falling weight.

(3) *Wearing Tests*, or those made for determining resistance to abrasion and impact, as in the case of paving-materials.

With reference to the character of the test specimen we have—

(1) *Specimen Tests*, or those made upon specimens of the material, specially prepared and given standard forms and dimensions.

(2) *Structural Tests*, or those made on full-sized structural forms, as floor systems, bridge members, brick piers, pipes, wire ropes, chains, riveted joints, etc., or on the structure as a whole, such as boilers, simple trusses, frames, and various parts of machines.

Complete standard rules for making tests of structural materials can be adopted for making all kinds of tests on specially prepared specimens, but they can be only partially prescribed for tests of structural forms.

#### THE ACCURACY OF MACHINES AND APPARATUS

**100. Methods of Determining the Accuracy and Sensitiveness of Testing Machines.**—General considerations of the accuracy, sensitiveness, and conditions which should obtain in testing machines have been given in Ch. II. Since it is of vital importance that the user of a testing machine should know approximately at least the accuracy of his machine, a brief statement of methods commonly used for testing accuracy and sensitiveness will here be made.

In testing the accuracy of vertical static-load machines five to ten equal increments of dead load can be placed on the platform of the machine, or on extensions formed by I-beams, and the corresponding readings of the weighing device compared with the known loads. After each incre-

ment of load has been added the sensitiveness may be determined by finding the additional weight which is required to make a perceptible indication on the weighing device. This method, even when pig iron is used, is too cumbersome and laborious for loads above 10,000 to 20,000 lb. In calibrating machines of the lever type, loads of this magnitude sufficient to determine the multiplying factor for the lever system and, assuming this factor constant, a correction coefficient applicable to the range of machine may be determined. It is not safe, however, to estimate the sensitiveness at high loads to be proportionately the same as at low loads. In calibrating hydraulic machines in which there is friction at stuffing boxes and around the piston, it is desirable to test the machine to full capacity.

Standardized calibrating levers with weights in capacities up to 100,000 lb. can now be purchased from manufacturers of testing machines. These furnish a more expedient and easy means of calibrating over a greater range of loads than the dead weight method. It is often possible to calibrate a very large testing machine by placing a smaller machine of known accuracy and sensitiveness on the weighing table of the larger machine and loading it. Hydraulic jacks of known accuracy may also be used for such purposes.

The most inexpensive device for calibrating large testing machines up to full capacity is the calibrated tension bar or compression prism equipped with a permanently attached extensometer. An annealed bar of high elastic limit and of such cross-section that the capacity of the machine will be well within the elastic limit of the bar is a desirable calibrating apparatus. The extensometer should be preferably of the self-indicating type, measuring deflections on two or more sides of the bar and reading to at least 0.001 in. It should be permanently attached to the bar and suitably protected from injury. The gage length should be sufficient to permit the least reading of the apparatus to correspond to a change in loading of 0.2 per cent of the capacity of the machine. The bar should be so secured in the jaws of the machine that an axial load is insured, and slipping prohibited.

If a compression prism is used the ratio of length to least radius of gyration should not exceed 20, and if a hollow cylinder be employed the ratio of thickness to diameter ought not to be less than 1:8. The average unit stress at full capacity of the machine should be less than two-thirds of the elastic limit. The quality of the material, gage length of compressionometer and its characteristics should conform to the corresponding specifications for the calibrating bar. Spherical seats or other means for securing an axial load should be used.

Calibrating test-pieces should be standardized at the Bureau of Standards or on machines of known calibration. Care should always be taken

to standardize such a test-piece under the same conditions as will surround the specimen when it is used for calibration purposes. A half dozen or more increments of load may be applied to the calibrating specimen at a speed not to exceed 0.1 in. per minute and corresponding readings of the deformations taken. After each increment of loading, the dead weight required to produce a movement of the weighing device furnishes a measure of sensitiveness.

Pressure gages on hydraulic machines should be frequently calibrated throughout the entire range of loading. If the pressure in the jack is measured to determine load on machine, calibrations under increasing and decreasing loadings should be made with the piston at different positions in the stroke. By so doing variations in the frictional resistances can be determined.

**101. The Calibration of Apparatus for Measuring Deformations.**—For certain classes of testing, a knowledge of the accuracy and sensitiveness of apparatus used to measure deformations is of as great value as similar information concerning the testing machine. For most purposes an accuracy of 1 per cent is sufficient in such apparatus. Essential considerations for different types of deformation apparatus have been discussed in the preceding chapter. When conditions permit the calibration of such devices may well be left to standard laboratories such as the Bureau of Standards or the Watertown Arsenal. If calibration is done in the home laboratory, the following method serves for a rough test of accuracy and sensitiveness.

The apparatus may be attached to a steel test-piece for which the stress-deformation curve has been accurately determined; and the test-piece gripped in a standard manner by the machine, so that slipping and improper distribution of stress are avoided. Increments of load are very slowly and uniformly applied to the test-piece and the corresponding readings of the apparatus taken. After each increment of loading one may determine sensitiveness by observing the increase and decrease in load required to produce a readable change on the deformation apparatus.

If a standardized deformation apparatus is at hand, it may also be attached to a specimen under conditions similar to those surrounding the apparatus which is being calibrated. The unknown device can then be compared with the standard under loading conditions indicated above.

A more accurate method than the above for calibrating extensometers and compressometers consists in clamping one end of the apparatus to a dummy specimen held in a lathe chuck and the other end to the centering spindle of the tail-stock. By using a microscope with a standardized micrometer eyepiece the movements of the pivot end of the spindle can be determined and compared with readings of the motions registered by the deformation apparatus. If a precision lathe can be had,

one part of the extensometer may be attached to the spindle on the tail-stock and the other to a specimen held on the lathe carriage. In this set-up the axes of specimen and spindle should be colinear. The lathe may then be set so that the carriage travels .02 or .01 in. per revolution of the chuck and a comparison of the deformation apparatus with the lathe screw determined.

Using a standardized linear dividing engine, reading directly to 0.002 mm. or less, still greater refinement may be obtained. With such apparatus it is possible to calibrate scales, micrometers, and practically all of the devices used in measuring deformations.

When calibrating apparatus in which accuracy in fabrication is based directly or indirectly upon the accuracy of a screw, readings of say one-tenth the range should be taken over the entire range of the apparatus to determine the cumulative error of the device. Periodic errors may be ascertained by a large number of readings of small increments of motion over a limited portion, say one-tenth, of the range. Furthermore, to eliminate periodic errors in the calibrating device it is essential that the apparatuses under comparison be shifted several times and the entire range and partial range calibrations repeated.

To avoid errors due to lost motion in screw-calibrating devices one must always approach the desired reading from the same direction. To accurately detect lost motion or lag and to determine the sensitiveness of the deformation apparatus, observations on minute forward and back motions of the moving part of the apparatus may be made under a microscope provided with a micrometer eyepiece.

## SELECTION AND PREPARATION OF SPECIMENS

**102. Selection of Specimens.**—It must be recognized at the outset that specimens are selected for testing with either of two objects in view—to compare the mechanical properties of certain materials or grades of the same material, or to ascertain the influence of certain conditions of fabrication, treatment and usage on the mechanical properties. In comparing mechanical properties the size, shape, method of fabrication, and subsequent treatment of the specimens are generally standardized, but in ascertaining the effects of structural conditions one or more of these is made variable. When choosing specimens for any kind of test the inspector must constantly bear in mind that the test results are valueless unless the specimens are truly representative of conditions and properties under investigation.

The numerous specifications of the American Society for Testing Materials, American Society for Mechanical Engineers, the Society of Automobile Engineers, American Railway Engineering Association, and

others, in general, cover the method of selecting test specimens of metals and metal elements which are to be used for various purposes. Test-pieces may be cut from the finished casting or rolled product. If the quality of metal is to be ascertained, specimens of cast metals should be separately poured into vertical dry molds. If specimens are cut from castings, it must be remembered that shrinkage strains exert a pronounced effect upon strength at all corners and angles in the casting and that the outside, especially in cast-iron members, is often much stronger than the center. When specimens are cut from rolled structural shapes, one should consider that the metal in the thin parts is harder, tougher and stronger than that of the thick portions which has received less work under the rolls.

With wrought iron a great difference will be found in specimens cut with and across the direction of the rolling, the former having much higher strength and a greater ductility. In steel plates there is little difference, and in rolled brass and copper plates there is no difference. In the case of the bronzes it is necessary to have test samples poured from different parts of the same melting, as the mixture changes its characteristics rapidly when in a melted state.

Piles of brick or building tile may be subdivided into small piles of approximately 100 each and one sample representing each small pile chosen. If the total number of samples is more than required the number may be reduced in similar manner. In selecting samples, color, depth of kiln-mark, number and position of checks and spalls, and ring under the hammer should be considered. If the sampling is done at the kiln, the position of the specimens with respect to the entrance and exit of the source of heat must be considered.

When samples of stone are being selected, it is necessary to secure specimens from the different strata which are being worked. If the surface has been exposed to the weather for a considerable time specimens should be cut from the interior. The faces of specimens should be referenced with respect to the rift in the rock. Specimens should not be selected from portions of rock adjacent to blast holes.

In selecting timber specimens, the rate of growth of the tree as told by the annual rings, position in the tree, the proportion of heartwood and sapwood, the proportion of spring and summer wood, the moisture content, the method of seasoning and the character and position of defects must all be considered.

**103. The Preparation of the Specimen.**—In order that the specimen may fairly represent the material under examination, or the particular plate, or bar, or rolled form from which it is to be taken, it is necessary to observe a number of rigid requirements.

The specimen must be obtained by cutting it out in a way that will

leave it perfectly straight. If a metal test-piece is bent in getting it out, it should be heated to straighten it; but this may often change the original molecular arrangement, and should be avoided if possible. When the specimen is cut from a larger portion of a plate or rolled form by shearing, it will invariably take a curved form. In this case *the plate, or form, should be sheared away from the specimen*, in narrow slices, so as to leave the test specimen unbent. If the specimen is bent and then straightened, it raises the elastic limit and hardens the metal, the same as any other kind of cold working. Instead of shearing, some milder process, such as planing or drilling or sawing, should be resorted to to obtain the test specimen. For, besides the bending action on the bar as a whole, the effect of the shearing or punching is to seriously injure the metal for about an eighth of an inch beyond the sheared surface, leaving it so non-ductile, or brittle, that it will not elongate appreciably, and hence under a tensile test these surfaces will be severed very early in the test, and the cracks so started may cause the remainder of the cross-section to tear asunder in detail. To prevent this action on sheared or punched specimens, at least an eighth of an inch of thickness should be removed from all punched or sheared faces, by reaming, planing, or filing.

Final finishing of hard metal specimens should be done with a file in order to avoid the torn and bruised surface conditions which result from the use of lathe and planer tools. Soft metal test-pieces should be finished with emery cloth. If the skin is removed from a casting by planer or milling device, it is well to remove the rectangular corners with a file in order that incipient cracks or irregularities caused by the tools may be eliminated. If soft metal specimens must be straightened, wooden or copper mallets should be employed; a steel hammer should never be used. The ends of metal compression specimens should be accurately ground to parallel plane surfaces.

To avoid the inclusion of material which may have been weakened in quarrying, it is necessary to saw compression test-pieces of stone from the interior of blocks somewhat larger than the test-pieces: roughing out the specimen with hammer and chisel may cause a large reduction in strength. If the latter method is permitted, care should be taken to make the sides of prismatic specimens plane. If accurate results are desired, it is well to grind to true planes the surfaces which are to be subjected to pressure. Specimens of brick, building tile and concrete will show greater strength if similarly treated. Since the expense of this work is often prohibitive, bedments such as are described in Art. 76, 77 and 78 are often used to overcome the effects of surface inequalities.

## TENSION TESTS

**104. Significance of Tension Tests.**—Tension tests are more common, more readily made, and more useful in revealing the true character of a metal than any other kind of mechanical test. In fact, when other kinds of tests are made it will commonly be well to accompany them with a few tensile tests for the purpose of being able better to co-ordinate the results with those obtained on other materials by similar tests, or on like materials by different tests. In this connection, however, it is well to remember that all metals are wanting in strict homogeneity, and that they may be regarded as aggregations of more or less dissimilar elements embedded in a common matrix, somewhat like granite. (See Art. 623 and 663.) For instance, the planes of rupture will be different for different kinds of tests on the same specimen, and hence the strength developed will be that of a different combination of elements in each case. Also, the strength to resist various kinds of stress may lie in entirely different elements of the aggregation, as, for instance, in the case of cast iron the strength to resist tension is the strength of the graphitic carbon matrix in which the iron crystals are embedded, while the strength in compression is largely strength of the iron crystals themselves.\*

What we call the maximum strength of the material, therefore, or its strength at rupture, is not usually the sum of the maximum resistances of the several elementary portions of the cross-section, since they do not all distort equally. It is often the case that actual rupture occurs successively over many elementary portions of the broken section before the final failure occurs. More especially is this true of the elastic limits of the material, while with iron and steel castings this failure in detail is so prominent as to cause the stress-diagram to be a curve almost from the beginning of the loading. Here, too, the irregular shrinkage often leaves very great internal stresses in the body, which causes some portions to come to their elastic limits and ultimate strength much earlier than others, again giving rise to a curved stress-diagram.

The tension test is especially well calculated to show what local irregularities may be found in a finished product, and to indicate to what extent the work of forging (rolling or hammering) has produced that degree of homogeneity expected of it.

The tension test is more readily standardized than any other so as to be independent of "personal equation" and of variations in the testing-machines employed. It also demands the least amount of preparation of the test specimen, if tests are to be made only for commercial purposes. Except for the inherent want of uniformity or of homogeneity mentioned above, therefore, the tension test may be made to give typical and uniform

\* M. Osmond.

results, and it should be considered as the best single test to make on any of the metals.

#### COMMERCIAL TENSION TESTS

**105. Object.**—In routine testing of metals under tension the ultimate strength, yield-point and per cent elongation are always determined and the character of the fracture noted. Frequently, the per cent reduction in area is also found. From these properties the static tensile strength, the limiting working unit stress, the ductility, and the homogeneity and character of structure of the metal is judged.\* Such tests, therefore, which can be performed with great speed and at low cost, serve a very useful purpose.

**106. Types of Tension Specimens.**—Experiments have shown that the form of a tension test-piece has an influence upon both the strength and

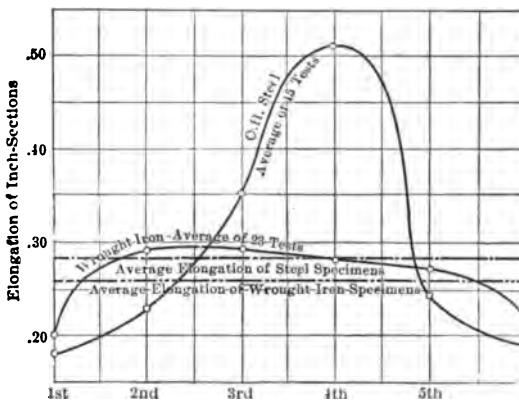


FIG. 1.—The Variation in the Distribution of the Elongation of the Several Inch Spaces of Six-inch Test Bars of Steel and Wrought Iron 0.56 In. in Diameter. (*Tests of Metals*, 1890.)

elongation. The influence on strength of grooving and sudden contractions in area is considered in Arts. 719 to 721. Fig. 1 shows the variation in unit elongation for successive spaces on steel and wrought iron bars. Fig. 2 illustrates the influence of the length of the gaged position on per cent elongation.

Tetmajer proposed that the elongation due to the neck be eliminated by subtracting the elongation in a 4-in. from the elongation in an 8-in. gage length and dividing by the difference in the gage lengths. The function, thus obtained, would be independent of the gage length of the specimen. This proposal has not, however, been widely adopted.

\* The character of structure in ductile metals is much better ascertained from tests on specimens around which a V-shaped groove has been turned. With such specimens the lines of stress are concentrated at the bottom of the groove and a square break revealing the character of the crystalline structure ensues.



On account of the above considerations it has become quite necessary to prescribe certain types of specimens for commercial tests in which the grading and acceptance of material is involved. M. Barba \* has shown that the resistance and per cent elongation remain constant provided the *relative* dimensions are not changed and the method of gripping and loading are identical. As the result of his work and a large number of tests, the French Commission adopted the relation  $l^2 = 66.67A$  or for cylindrical specimens  $l = 7.2D$ , where  $l$  is the measured length on which the elongation is computed,  $D$  is the diameter, and  $A$  the area of cross-section. The German Commissions use  $l = 11.3\sqrt{A}$ , which is equal to  $l = 10D$  if the

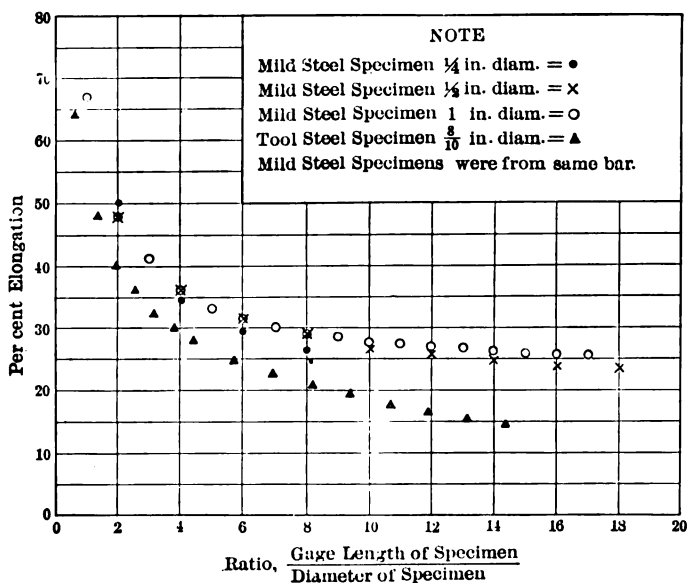


FIG. 2.—The Influence of the Ratio  $\frac{\text{Gage Length}}{\text{Diameter}}$  on the Per Cent Elongation of Steel Specimens.

specimen is cylindrical. Engineering societies in the United States have not rigidly adhered to either of these rules, or even Barba's law, in standardizing the shape of test-pieces cut from plates or other rolled sections. However, they have prescribed fixed standards for cylindrical and flat specimens (see Figs. 3c and 3e). For flat specimens over  $\frac{1}{4}$  in. thick, the gage length is 8 in. and the width  $1\frac{1}{2}$  in. The width of specimens cut from material less than  $\frac{1}{4}$  in. in thickness must be five times the thickness with a minimum of  $\frac{3}{4}$  in., and the gage length must be twenty-four times the thickness with a minimum of 2 in.

\* *Mem. de la Société des Ing. Civils*, 1880, I, p. 682. See also Unwin's *The Testing of Materials of Construction*.

In Fig. 3 are shown seven types of specimens dimensioned in accordance with practice. For rough tests on ductile rounds or flats, form (a)

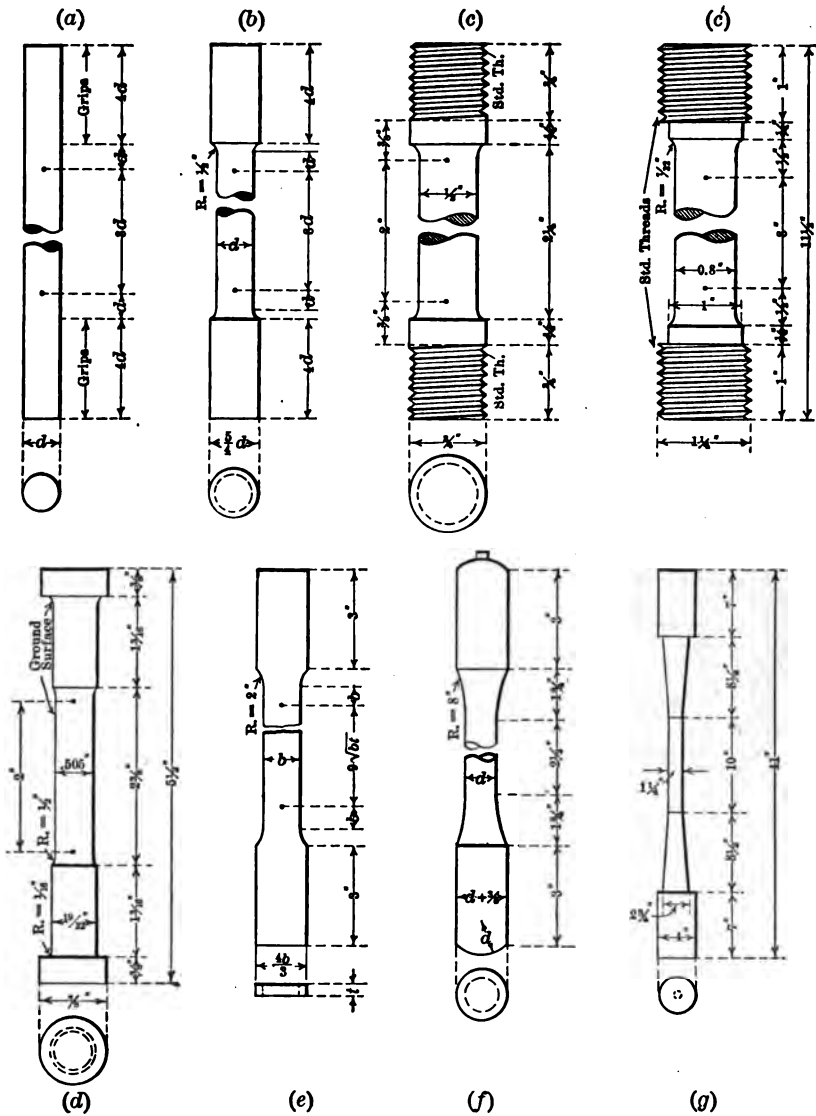


FIG. 3.—Types of Specimens for Tensile Tests.

is suitable. Form (b) is a more expensive specimen which is less liable to slip in the grips and also receives a lower intensity of compression from the grips than does form (a). For more careful tests in which it is highly

desirable to avoid slipping at the grips, form (c) or (c') is used in conjunction with the holder shown in Fig. 29, Art. 69. Automobile steels of very high elastic limit and great hardness cannot be readily fashioned into form (c) after hardening. If so fashioned before heat treatment, these steels are liable to become warped and weakened at the screw threads. For such steels form \* (d) may be used in connection with the holder shown in Fig. 30, Art. 69. An inexpensive casting which makes a very satisfactory test-piece for rough tests on cast iron, mal-

leable iron and similar brittle metals is form (f). Form (g) has been used with success by Prof. W. H. Warren of Sydney University, Australia, in testing the tensile strengths of woods.

**107. Testing.**—The dimensions of cross-section of the specimen are measured at several places along the gage length. If the test-piece is cylindrical, mutually perpendicular diameters should be measured at each cross-section. Measurements of metal should be accurate to  $\frac{1}{10}$  of 1 per cent. The average cross-section of deformed bars can be computed from determinations of weight and length.

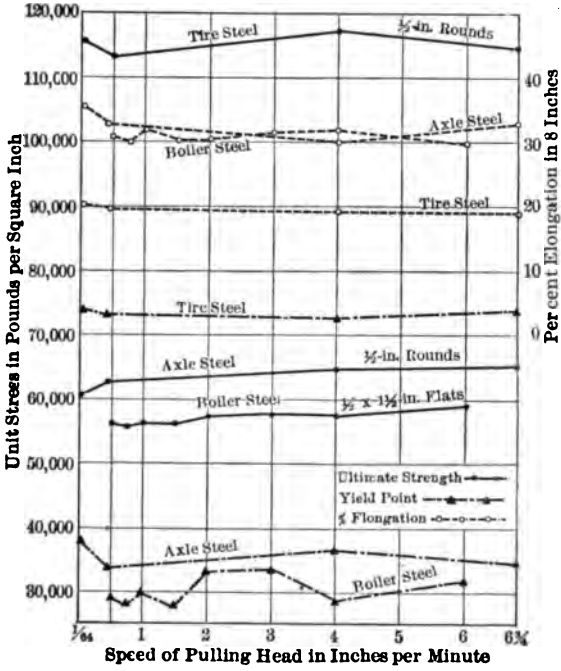


FIG. 4.—The Effect of Speed on the Tensile Strength and Elongation of Steel. (Rept of Com. O, A.S.T.M., see *Proceedings*, 1906.)

The specimen is then placed in a V-block and punch-marked at intervals of 1 inch along the entire gage length. A laying-off gage or a multiple punch is a time-saver in this operation. Fine wire and other small sections which would be seriously weakened by punching may be marked with ink.

For variations in speed of the pulling head of the testing machine up

\* See description of this type of specimen given by K. W. Zimmerschied in *Trans. Soc. Auto. Engr.*, Vol. 8, pt. 2, p. 161; also see Marten's *Handbook of Testing Materials*, p. 10.

to 6 in. per minute, there is no pronounced effect on the mechanical properties of soft steel as Fig. 4 shows. The higher carbon steel specimens show practically the same properties for speeds less than 3 in. per minute. The greatest effect is produced on the yield-point. However, if one wishes to accurately determine the yield-point a speed of 2 to 4 in. per minute may be applied until about three-fourths of the estimated yield-point load is reached, the speed then may be quickly cut down to about  $\frac{1}{4}$  in. per minute, the yield-point determined, and the high speed again reapplied to rupture. For cast iron and similar brittle materials the speed throughout the test ought not exceed 0.02 or 0.03 in. per minute per inch of gage length. The speed should be constant for any given series of tests on brittle plastic materials.

The French Commission recommends that tests be continuously progressive; that the duration of test be proportional to the volume of the specimen (one to six minutes for ordinary sizes of specimen, less than thirty seconds for test-pieces under 0.2 in. thick); that heating of the bar must be avoided, especially with the softer metals.

In tests of high carbon steels and in tests in which there is slipping at the grips a pair of Capp's multiplying dividers (see Art. 95) will be found useful.

**108. Observations for Record.**—The record should contain sufficient information so that the history of the specimen previous to the test may be traced. In inspection at the mill this includes heat number, specimen number, and such other information as may be needed to reference the specimen to the portion of the heat or to the member from which it was taken.

The first sign of weakening at yield-point of the specimen should be carefully ascertained. In wrought iron and the low-carbon steels this is readily determined by the drop of beam, by the rapid increase in motion of the divider pointer and, in rolled material, by scaling. If rolled bars of uniform cross-section are used, the scaling will appear first at the grips, owing to the combined stress existing there, and gradually extend toward the center of the specimen. It will be noted that the scaling advances on lines at about 45 degrees with the axis of the specimen: i.e., on the surface traces of the planes upon which maximum shear stress exists.

The maximum load is next determined. It will be found to occur simultaneously with the commencement of the "necking down" action in ductile materials, with rupture in brittle materials. At present it is not customary to record the actual breaking load for ductile materials, since it is not regarded as an important index of strength. After rupture the test-piece is again laid in the V-block with the fractured ends in contact and a record of the length of gage across break is made.

If the specimen has a "cup and cone" fracture, it will be found easier to measure the reduced diameter on the lips of the cup than on the cone. The reduced areas of rectangular specimens can be most readily determined by measuring  $b$ ,  $d_1$  and  $d_2$  as shown in Fig. 5. To measure fractures of irregular outline, a micrometer provided with a conical spindle and anvil will be found convenient.

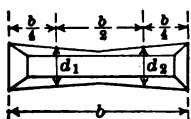


FIG. 5.—Reduced Area of a Rectangular

$$\text{Test Piece} = b \frac{(d_1 + d_2)}{2}$$

Two characterizations of fracture are generally made, one with reference to shape and the other with reference to texture. For example, mild steel fractures are commonly "cup and cone" in shape and "silky" in texture; hard steel fractures break squarely across—"square break"—and are more or less finely crystalline in texture. Fig. 6 shows the fractures commonly observed in metals and suggested

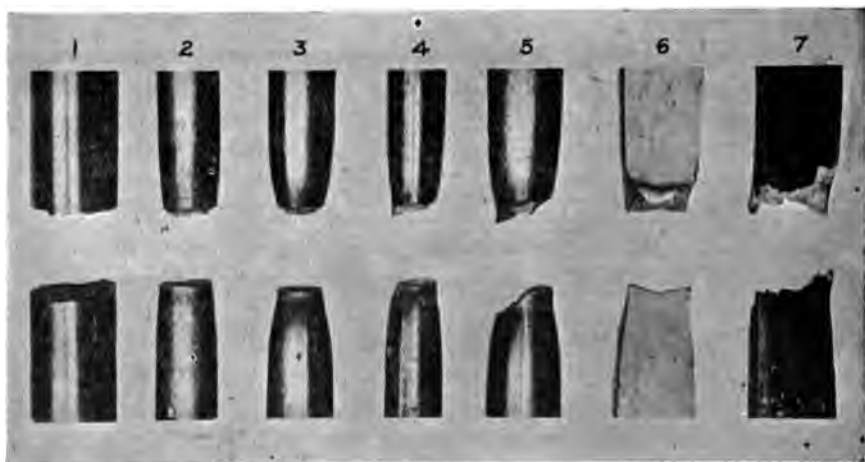


FIG. 6.—Characteristic Fractures of Ferrous Metals in Tension.

- (1) Common in cast iron, designated as square break and fine or coarsely crystalline; also in very high carbon steel, texture finely crystalline.
- (2) Common to high carbon steel, called fin cup or flat cone with granular to crystalline texture.
- (3) Common to soft and medium carbon steels, called full cup and cone with texture silky.
- (4) and (5) Common in soft and medium steels (especially when eccentrically loaded), designated as three-quarter cup and cone (4), and half cup and cone (5).
- (6) Common in soft and medium steel bars of flat or square section, designated sheared cone with silky texture.
- (7) Common in wrought iron, designated jagged and fibrous. (Overheated soft or medium steel may also present a jagged break but not fibrous.)

characterizations of the same. Unusual features in the fracture should always be recorded and their causes, if possible, ascertained. For the latter purpose the microscope is a valuable aid.

## EXTENSOMETER TENSION TESTS

**109. Object.**—In addition to the objects mentioned under commercial tension tests, the extensometer test affords a determination of the stress-elongation curve for the material. Consequently the elastic limit or the limit of proportionality and the resilience may also be measured.

**110. Testing.**—Specimens like form (c) or (d), Fig. 3, are preferable for extensometer tests; but it is desirable that the gage length for the apparatus should be at least 8 in. For steels, a specimen fashioned as indicated with a diameter of 0.798 in. (area 0.5 sq.in.), length between shoulders of 9 in. and a gage length of 8 in. is convenient and satisfactory.

To avoid errors arising from the bending of the specimen due to eccentricity in loading, non-homogeneity or initial curvature in the specimen, extensometers provided with three-point contact at each collar are preferable to extensometers of same type having two-point contact. The accuracy of the extensometer should be commensurate with the magnitude of the deformation to be measured. Ordinarily the least reading of the apparatus should be less than one one-hundredth of the deformation at the elastic limit. For steel this requires apparatus reading to 0.0001 in. General requirements for extensometers may be found in Art. 79.

During the test the speed of the pulling head should be very low, not over 0.01 in. per minute per inch of gage length, and the machine may be stopped long enough to secure readings of deformation. It is of great importance to avoid vibration or shocks during the application of the load, since such disturbances greatly impair both the accuracy and sensitiveness of the extensometer.

There are two methods of loading which may be used, depending upon the nature of the material and the information desired. If the limit of proportionality is to be determined the load is progressively applied in increments equivalent to about one-tenth of the estimated value of that stress and the corresponding deformations observed. When a stress approximating the limit of proportionality is reached the increments are reduced to about one-tenth of their former value until the yield-point has been passed. It is customary to remove the extensometer from the specimen after passing the yield-point in order to avoid injury to the instrument. Subsequent measurements of elongation may be made with a pair of dividers.

The second method of loading is more often adopted with materials having a curvilinear stress-deformation diagram than with iron or steel. It consists in determining the maximum total deformation and set corresponding to repeated applications of each load. An initial load corresponding to one-twentieth the ultimate strength of the material, or thereabouts, may be applied and removed with determination of accompanying

deformation and set. If set occurs the load is repeated and readings of deformation and set taken until the set becomes constant. Then the load is doubled and the same cycle of operations is repeated. This process is applied again and again until the load is reached at which the set continually increases. When this method is applied to a material having a true elastic limit it is well to decrease the increments of load in proximity of the elastic limit.

**111. Stress-deformation Diagrams.**—It is customary in this country to plot curves using unit stress as ordinates and unit deformations as abscissas with the curve lying in the first quadrant. The scales for such diagrams should be so selected that the slope of the curve will lie between 30 and 70 degrees with the horizontal; otherwise the curvature of the diagram is made too flat or too steep and there is also difficulty in determining the modulus of elasticity with accuracy. If the repetition method of loading has been employed in testing, curves showing the gross sets, gross deformations, and net deformations are generally plotted against unit stress. On such diagrams in addition to a suitable title and proper labeling of the coordinates, it is good practice to indicate the value of the modulus of elasticity beside the line from which it was gotten, to indicate the elastic limit or limit of proportionality, the yield-point, and ultimate strength. The diagram is, in this way, made to furnish the most essential information secured in the test.

### COMPRESSION TESTS

**112. Objects of Compression Tests.**—Whereas tension tests are made for the purpose of determining many of the more significant mechanical properties of materials which are more or less ductile, compression tests are made chiefly to determine resistance to compression and the elastic properties under compression.\* In testing materials possessing a high degree of elasticity the elastic limit, yield-point, and modulus of elasticity may be gotten. The determination of the ultimate strength is dependent upon the plasticity of the metal after the yield-point. In the softer varieties of steel there is no well-defined point in the loading at which a complete disintegration of structure takes place. From tests of columns made of such materials it appears that the ultimate strength is limited by the elastic limit. Consequently it is quite common to regard the elastic limit as a measure of ultimate strength in compression for these steels.

Materials, possessing a high degree of plasticity, like the minor metals and their alloys, have poorly defined elastic limits and the compressive strength is often based upon the load sustained at a given unit deforma-

\* Herein we shall refer to compression tests as practiced on short prisms. The testing of columns involves consideration of conditions of fabrication, end restraint, form and position of load in addition to the characteristics of the material itself.

tion. *These values furnish a basis of comparison but are far from criteria of structural strength unless determined from long-time applications of load.*

For brittle materials like concrete, building tile, stone, brick and timber, the compression test is of most value in establishing criteria of mechanical properties of materials. In tests on these substances, the unit stress at first crack or first sign of failure, at elastic limit—if there is one—and at ultimate are found. The position of the first crack, the character of the explosion at rupture and the shapes of the fragments are all noteworthy. The unit stress at first sign of failure coupled with a knowledge of the place of initial weakening may indicate faulty imposition of load or a local defect in material. The character of the explosion at rupture and the shape of the fragments are also of assistance in determining whether the load was axially or eccentrically applied. Flowing of the bedment often produces vertical splitting of the test-piece.

**113. The Form of Compression Specimens.**—The form of specimen which has been most frequently adopted is the cube. For materials which rupture on planes inclined more than  $45^\circ$  with the horizontal, the cube is not suitable; since the strength is increased by frictional restraint acting at the surfaces under pressure. Prisms or cylinders with a height equal to twice the least lateral dimension are better types of test-piece for such materials. Owing to weaknesses at corners due to the methods of fashioning the specimen and to the impossibility of securing full resistance from the material in the corners, a cylindrical test-piece is preferable.

In Fig. 7 appears three curves showing the relation of crushing strength to the ratio  $\frac{\text{height}}{\text{least lateral dimension}}$ . The results on cast-iron cylinders are digested from tests by Mr. Chas. Bouton.\* For these tests over 100 specimens were prepared from five bars of each of two kinds of cast iron. Comprehensive tests on Swiss sandstone prisms by Prof. J. Bauschinger † furnished the data for the corresponding curve in the above figure.

From these tests on rectangular prisms, Bauschinger derived the following formula,

$$S_c = 5600 + 1400 \frac{A}{h};$$

in which  $S_c$  is in pounds per square inch,  $A$  is the area in square inches, and  $h$  is the height in inches. For a general formula he recommended

$$S_c = \sqrt{\frac{\sqrt{A}}{u}} \left( a + b \frac{\sqrt{A}}{h} \right),$$

\* M.S. Thesis, *Theory and Experiments on Laws of Crushing Strength of Short Prisms*, Washington Univ., 1891.

† *Mittheilungen aus dem Mechanisch Technischen Laboratorium der K. Technischen Hochschule in München*, von J. Bauschinger, Vol. 6, 1876.



where  $u$ =perimeter of cross-section;  $a$  and  $b$  are constants, and the other quantities are the same as before. A simpler relation for Bauschinger's tests on sandstone is given by

$$S_s = 5500 + 1565 \frac{d}{h},$$

where  $d$  is the minimum lateral dimension.

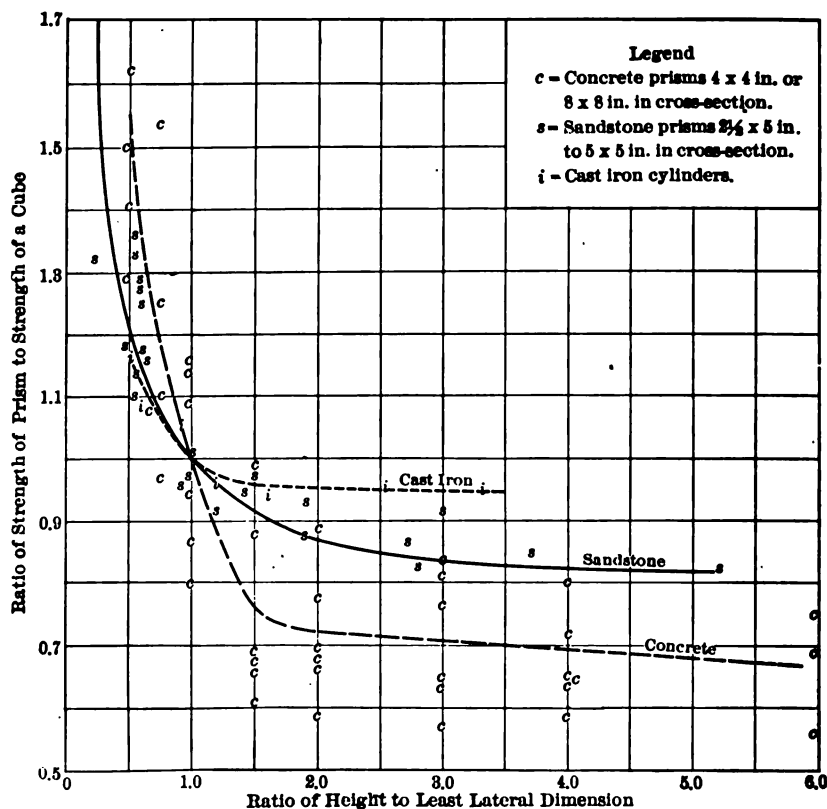


FIG. 7.—Relations between the Crushing Strengths of Prisms and Cubes.

The data for the third curve in Fig. 7 was obtained from 192 tests at several college laboratories in cooperation with the National Association of Cement Users.\*

From the equation for the curve representing tests on sandstone prisms, Fig. 7, the following relation appears;

$$\frac{\text{Strength of prism}}{\text{Strength of cube}} = 0.788 + 0.222 \frac{d}{h}.$$

\* *Concrete-Cement Age*, Vol. 4, p. 141.

From this equation it appears that a sandstone prism having a height equal to twice its least width has 89 per cent of the strength of a cube of the same material. Whereas from the tests on concrete it appears that such a prism has only 73 per cent of the strength of a concrete cube. It therefore seems essential that a standard prismatic form be adopted for compression specimens.

The American Society for Testing Materials has specified for metals a cylindrical specimen between 1 and 1.129 in. in diameter and from 2.5 to 4 in. high.\* At present, sentiment in this country seems to favor

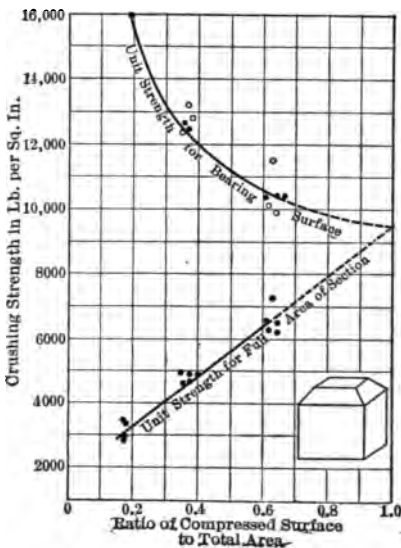


FIG. 8.—Crushing Strengths of Cubes with Chamfered Edges. (Bauschinger.)

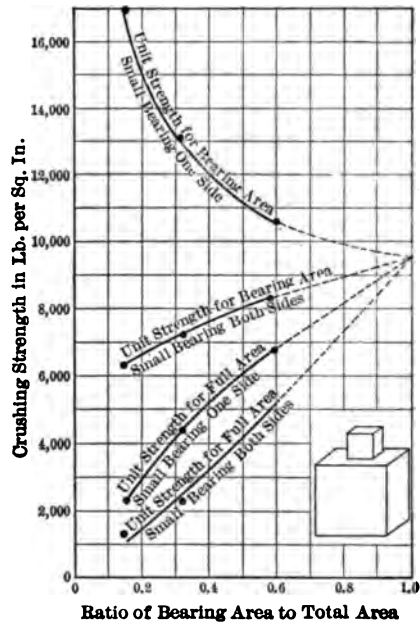


FIG. 9.—Effect of Loading a Portion only of the Surface of a Cube.

the adoption of a 6 by 12-in. cylinder for a concrete test-piece and a 2 by 4-in. cylinder for mortar.

In spite of theoretical considerations, however, the necessity of translating into new terms the mass of data which has been accumulated from tests on cubes is an obstacle to the adoption of any other form. Consequently, it is very probable that the cube, although improper in form, will still be used in tests of many materials when strength, alone, is wanted. For making tests of the modulus of elasticity, elastic limit, and yield-point on metals, the A.S.T.M.\* permits the use of cylinders ranging in length from 10 to 15 diameters.

\* See *Methods for Compression Tests of Metals*, E1-16, in *A.S.T.M. Standards*, 1916.

**114. Effects of Loading a Portion of the Cross-section.**—Tests by Bauschinger on the effect of chamfered edges on the strength of sandstone cubes gave the results shown in Fig. 8. The tests show that material symmetrically disposed outside of the bearing surfaces increases the strength of the test-piece. This increase is less than 3.2 per cent, however, if the bearing area is more than 80 per cent of the gross cross-section.

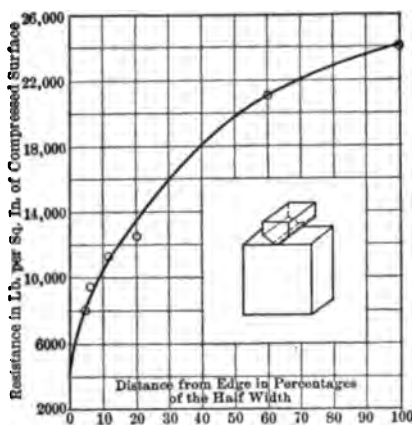


FIG. 10.—Effect of Loading a Zone on the Surface of a Cube. (Bauschinger.)

area when the center of the bearing area is 4 per cent of width of the cube from the edge is 9500 lb., the normal strength of a cube.

**115. Apparatus Required for Compression Tests.**—Descriptions of testing machines, bearing blocks, compressometers, and bedments will be found in Ch. II. For tests on the modulus of elasticity the compressometer should be accurate and sensitive to one one-hundredth of the deformation in the gage length. It should record deformations on at least two sides of the specimen. The yokes attaching the compressometer to the specimen should be placed not less than half of the diameter of the specimen from the nearer bearing surface. If the yield-point of hard steel is to be gotten, a pair of Capp's multiplying dividers will be found convenient. A spherical seat to permit adjustment due to non-parallelism of the heads of the specimen is especially desirable in testing brittle materials. The value of the seat is small, however, unless provision is made for properly centering the specimens with respect to it.

**116. Testing.**—The cross-section of the specimen should be determined at several points along its length. On cylindrical specimens measurements on mutually perpendicular diameters should be made. Metals should be measured to one-one-thousandth part and non-metallic materials to one part in five hundred. Building tile, if scored, should be

If the pressed surface is square and symmetrically located on a side of a cube the relations of resistances per square inch to the ratio of bearing area over total area are shown in Fig. 9.

The effect of loading a rectangular zone having a width equal to 5 per cent of the side of the cube and a length equal to the side of the cube was also studied by Bauschinger. In this case the resistance per square inch is a function of the distance of the zone from the edge of the surface. The results summarized in Fig. 10 show that the resistance per square inch of bearing

measured outside of the scoring and no allowance made for the area of fillets. Although in specifications for all hollow building block and tile it is customary to demand a minimum strength in terms of the gross section only, it is worth while to obtain the net area in order that the strength of the material itself may be judged.

Great care should be exercised to adjust both the bearing block and specimen so that the line of pressure will pass through the axis of test-piece, bearing block, and testing machine.

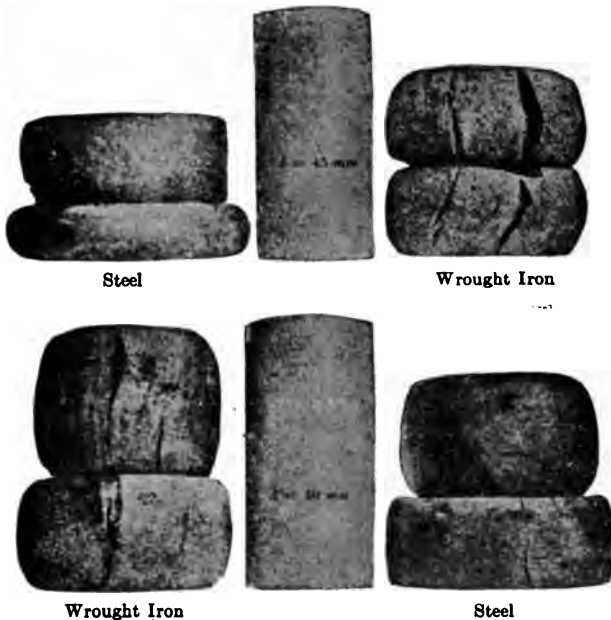


FIG. 11.—Relative Malleability of Wrought Iron and Soft Steel. All the specimens were originally of the shape of the one remaining undeformed. The wrought iron specimens uniformly show large cracks. (From von Tetmajer's *Communications*, Vol. 4, Pl. 5.)

The speed of the movable head of the testing machine ought not to exceed 0.02 in. per minute per inch of height in compressing iron or steel specimens. For plastic and brittle metals, stone, concrete, clay products and wood, speeds should not exceed 0.005 in. per minute per inch of height of test-piece. Where strength only is desired the rate of loading may be made more rapid for loads less than half to three-fourths of the ultimate strength. For very plastic materials a much slower rate of loading should be used if a quantitative determination of the crushing strength is wanted. In any series of tests the method of applying the load should remain constant.

The manner of making compression tests with a compressometer is the same as that outlined in Art. 110. Also, the method of plotting the curve sheet and the information desired may be gotten from Art. 111.

In making a test for crushing strength only, it is well to surround brittle test-pieces with wire meshing to prohibit fragments from flying at rupture.



FIG. 12.—Bouton's Compression Tests on Cast Iron.

**117. Observations During Test.**—Care should be taken to determine the position and character of the first crack together with the load at which it occurs. With materials like low-carbon steel and wrought iron the yield-point will be denoted by the drop of the beam, by the rapid increase in motion of the divider pointers, and—in rolled material—by scaling. In tests of brittle materials the shape of the fracture should be stated thus: “pyramidal,” “plane inclined  $\theta$  degrees to horizontal,” or “cone”; and the texture of the broken surfaces examined and reported. Characteristic fractures of wrought iron, cast-iron and sandstone speci-

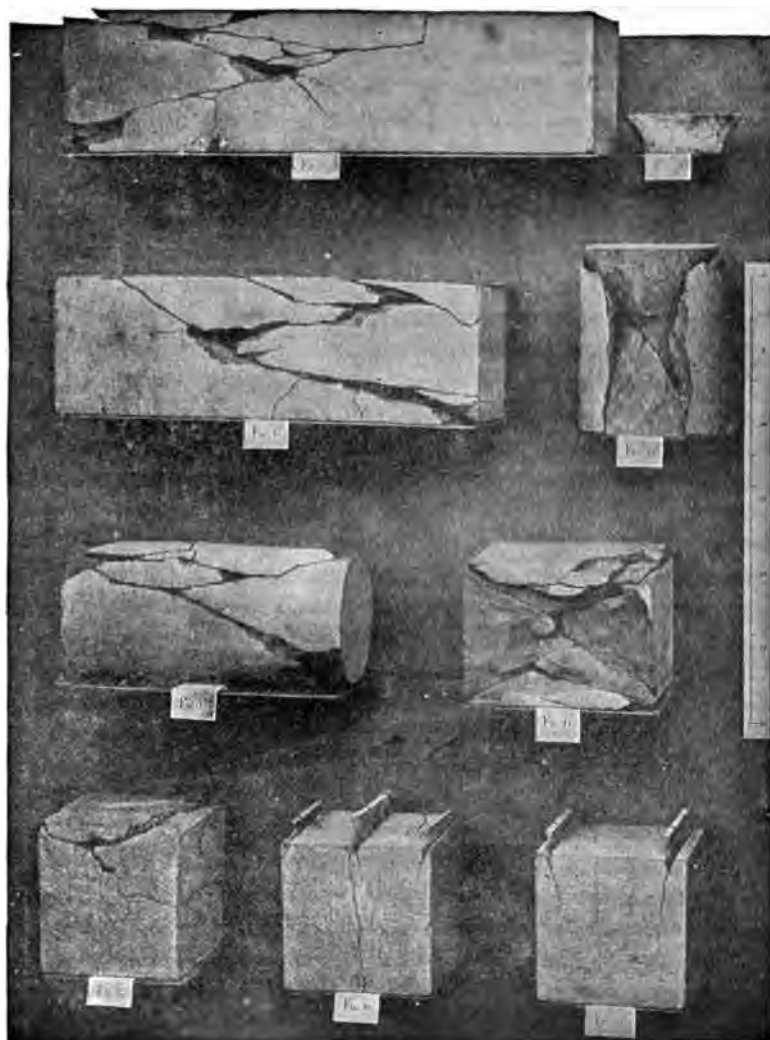


FIG. 13.—Bauschinger's Compression Tests on Sandstone.



FIG. 14.—Typical Failures of 6×12-inch Concrete Cylinders.

mens may be found in Figs. 11, 12 and 13, respectively. Typical fractures of concrete cylinders appear in Fig. 14.

### TRANSVERSE TESTS

**118. Objects of Transverse Tests.**—For determining ultimate strength, elastic limit, yield-point, resilience and modulus of elasticity of brittle materials in cross-bending, the transverse test is often used. The transverse is of especial value in determining the strength, stiffness, and toughness of brittle materials. Since the deflections of brittle specimens are many times larger than the elongation in tension tests, a much more accurate determination of resilience can be made in the transverse test without expensive apparatus for measuring deformations. Furthermore, both the machine and the specimen required are inexpensive and the test may be rapidly made.

Cast-iron, brick, stone, and concrete are tested principally for strength, sometimes the resilience and modulus of elasticity are obtained. Timber is tested for its strength, stiffness and resilience. Springs and spring-steel are tested for elastic limit, deflection under given loads, and resilience. Railroad rails are sometimes tested for elastic limit and ultimate strength. I-beams and other structural shapes used as beams are also tested to determine constants for use in design. Transverse tests are also made for scientific purposes to test the correctness of the ordinary flexure formulæ for strength and deflection.

In most cases transverse tests of ductile materials are not so well adapted to determine quality as tensile tests. Furthermore, the modulus of rupture and transverse elastic limit of such materials vary greatly with the length of span.

Since three kinds of stress, tension, compression, and shearing, are developed when a beam is bent under the action of external forces, the problem is more complex than those thus far considered. Usually the shearing stresses are left out of account in designing both for strength and stiffness, but the conditions under which this stress should be recognized and taken account of are given in Art. 28 for strength, and Art. 30 for deflection.

**119. Specimens for Transverse Tests.**—For cast metals, the A. S. T. M. recommends a vertically cast cylindrical specimen  $1\frac{1}{4}$  in. in diameter with a span of at least 15 diameters.\*

By employing a circular section, a uniform thickness of skin is ensured, and in unannealed specimens shrinkage strains due to corners are obviated. For malleable iron, however, the Society permits a test-piece of a rectangular section 1 in. broad by  $\frac{1}{2}$ ,  $\frac{5}{8}$  or  $\frac{3}{4}$  in. thick with a 12-in. span. As Fig.

\* See *Methods for Transverse Tests of Metals*, E1-16, A.S.T.M. Standards, 1916.

15 shows, the fixing of a standard span for such metals is of importance. The whole problem is now being considered by the I.A.T.M.

When tests are made to determine constants for I-beams, T-bars or similar sections, it is necessary that the specimen be geometrically similar (preferably of the same size to avoid differences due to rolling) to the section under investigation. Tests showing the variation in the transverse

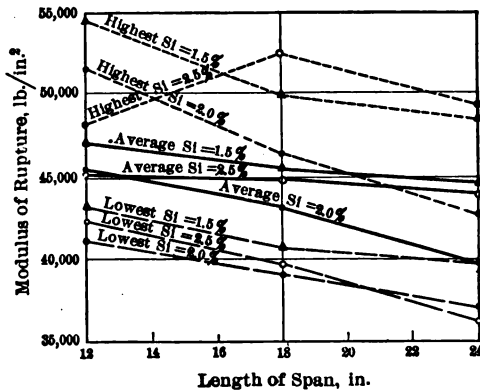


FIG. 15.—Relation of Span to Modulus of Rupture for Cast-iron Arbitration Test Bars Differing in Silicon Content. Each Average Represents Nine Tests. (Matthews, in *Proc. A.S.T.M.*, Vol. 10, p. 303).

strength properties, due to changing the shape but not the area of cross-section, appear in Table I.

For plate glass and flat springs, test-pieces should be flat with a length ten or more times the depth. For timber, rectangular specimens are best. The Forest Products Laboratory uses 2 by 2 by 30 in. specimens with a 28-in. span for small beam tests. In tests of larger timber beams it is advisable to make the span from fifteen to twenty times the depth. For cement and mortar specimens 1 by 1 by 6-in. prisms with a 5-in. span are convenient.

**120. Apparatus Required for Transverse Tests.**—Descriptions and general considerations of apparatus will be found in Ch. II. A testing machine of capacity equal to two to four times the estimated resistance of the test-piece should be selected. If a deflectometer is used, it should be accurate and sensitive to one one-hundredth of the maximum deflection, for tests involving this determination only, and to one-hundredth of the deflection at the elastic limit for tests on the modulus of elasticity. If deformeters are used to measure fiber deformations, they should have an accuracy and sensitiveness not less than one one-hundredth of the deformation in the gaged length when the fibers are stressed to their elastic limit. Care should be taken to avoid fastening deformeters close to application points of the loads.



TABLE 1.—THE EFFECT OF SHAPE ON THE TRANSVERSE STRENGTH OF CAST IRON

(C. H. Benjamin in *Machinery*, May, 1906)

Beams loaded at center over an 18-inch span

No. of Results Averaged.	SECTION.			Breaking Load, $P$ (lb.).	Modulus of Rupture, $S_m = \frac{4.5Pc}{I}$ , (lb./in. <sup>2</sup> ).	Modulus of Elasticity, in 1,000,000 lb./in. <sup>2</sup>
	Shape.	Area (in. <sup>2</sup> ).	$\frac{I}{c}$ * (in. <sup>3</sup> ).			
2	●	4.42	1.31	7,375	25,270	8.28
2	■	4.44	1.55	8,125	23,450	9.25
2	■	4.58	3.12	16,150	23,210	6.61
2	○	4.40	2.88	19,900	31,125	7.05
2	○	4.36	3.38	21,400	28,450	5.61
2	○	4.41	3.22	25,250	35,300	6.49†
2	□	4.38	5.11	28,175	24,840	4.41
2	⌈	4.56	5.78	24,250	19,400	4.54
2	⌈	4.84	6.46	31,550	22,010	5.62
2	⌈	4.61	6.52	31,750	21,940	4.71
2	⌈	4.88	6.48	34,625	24,060	4.84
1	⌈	4.51	0.81	5,400	30,000	10.74
1	⌈	5.10	1.99	8,350	18,900	8.57
1	⌈	4.61	0.69	4,700	30,580	10.17
1	⌈	4.80	1.61	8,800	24,600	11.06
1	⌈	4.41	0.83	4,400	23,700	
1	⌈	4.60	2.30	12,250	24,000	9.34
1	⌈	4.47	1.77	7,900	20,050	7.10
1	⌈	5.02	4.36	22,600	23,250	7.88
1	⌈	4.50	1.78	10,200	25,800	6.23
1	⌈	5.18	5.95	25,000	18,900	7.28

\*  $c$  is the distance to the extreme fiber in tension.

† Only one result.

Rocker supports or swinging links should be used at the ends of the specimen to prevent longitudinal compression in the lower fibers when the beam is loaded. Also, if a pair of loads is imposed through a loading beam, rollers should be used between specimen and loading beam to avoid compounding of the specimen and loading beam through friction at the load points. A high intensity of compression under the loads and over the reaction should be avoided by the use of metal bearing plates.

For transmitting loads to large timber beams, the Forest Products Laboratory uses heavy maple shoes which rest upon  $\frac{1}{8}$ -in. steel plates. The lower surface of the shoes is cut to a circular arc having a cord length of  $13\frac{1}{2}$  in. and a mid-ordinate of  $\frac{9}{16}$  in.

When structural shapes are tested, they should be braced and loaded as far as possible, in accordance with the conditions of use. I-beams and girders should, in most cases, be laterally braced at the supports.

For tests of cast and brittle metals, the speed of moving head is limited to 0.2 in. per minute by the A.S.T.M. The Forest Products Laboratory uses a speed of  $\frac{1}{4}$  in. per minute on large timber beams and 0.1 in. per minute on 2 by 2 by 30 in. specimens. For tests on brick, stone and similar brittle materials the speed should be such that the rate of deforming the extreme fiber of the specimen will not exceed 0.002 or 0.003 per minute.\* In the tests where deflections are being read, it is best to apply the load at a sufficiently low rate to permit the readings being taken without stopping the machine. Such procedure is impossible when deformeters are being used, but the time intervals allowed for readings should be constant during such tests.

It is well to use increments of load of about one-tenth the estimated load at elastic limit and when in the proximity of the elastic limit reduce the increments to one-fifth of their former value.

**121. Observations During Test.**—The transverse elastic limit of metals as determined from deflections is generally higher than the tensile elastic limit and not so plainly perceived. The yield-point is denoted by the drop of beam and by scaling in the case of rolled sections. The position of the scaling and its advance over the test-piece are well worth careful note. The ultimate strength of ductile metal specimens is often very hard to determine with exactness; but, if a slow speed is used, the maximum load may be approximately ascertained.

In testing brittle materials like cast iron, the observer must be alert if he is to note the load and deflection at failure. The character of the fracture should be recorded with care.

When testing timber, concrete, malleable cast iron and like substances, the load at first crack should be noted and the position and character of the crack recorded.

**122. Load Deflection Curves.**—Ordinarily, loads are plotted as ordinates and deflections as abscissas. The scales should be such that the initial portion of the curve has a slope of over 20 degrees with respect to the load axis. On such diagrams it is good form to indicate the elastic limit, yield-point, modulus of rupture ( $S_m$ ), modulus of elasticity, and resilience.

\* Formulas for speed of the moving head of the testing machine appear in Art. 231.

**IMPACT TESTS**

**123. Object of Impact Tests.**—For measuring the abilities of materials to withstand sudden shocks or blows, experience has shown that none of the static tests are entirely suitable. Such determinations can best be made under tests approximating the kind of loading which the material must bear. Although impact tensile and compressive stresses are commonly encountered in construction, the difficulty of providing suitable means for producing and measuring these stresses has prevented the adoption of tensile or compressive impact tests. Impact transverse tests have, however, been used to determine the shock resistance of structural elements like rails, axles, car wheels and car couplers, and to determine the shock-resisting qualities of both brittle and ductile materials. Of particular value is this test in ascertaining the influence of heat treatment on the brittleness of steels.

Owing to the impossibility of accurately measuring the proportion of the energy of a blow which is absorbed by the specimen, impact tests cannot give absolute indications of shock resistance. Nevertheless, by carefully standardizing machines, the methods of making tests, and the forms of specimens, it is possible to make comparisons which are very valuable.

In tests of structural forms, the acceptance of material is generally based upon the ability of certain selected pieces to withstand an arbitrary number of blows without being deformed beyond a certain limit. In tests of small specimens for the purpose of ascertaining the quality of the metal it is possible to compute with fair accuracy the energy of rupture which serves as a criterion of quality. The Forest Products Laboratory determines the energy of rupture per unit of volume, the elastic limit and modulus of elasticity of small timber beams under impact. The energy of rupture of ductile materials may be gotten if the shape of the specimen is suitable.

**124. Specimens for Impact Tests.**—For brittle castings round bars are well adapted for impact specimens. For cast iron and malleable cast iron, the specimens used in standard transverse tests may be employed. If parts are to be used in service as cast, the skin should not be removed from test-pieces.

To produce failure in ductile materials, it is necessary to make an abrupt contraction in area at the section under maximum stress. This may be accomplished by nicking, by sawing, or by drilling a hole to serve as the base of a saw-slot. Mr. C. Fremont who made exhaustive experiments to standardize the notched impact test on metals, was led to the adoption of a specimen 30 by 10 by 8 mm. with a 1 by 1 mm. notch extending transversely across the 10 mm. face. A committee of the International Association for Testing Materials appointed to investigate this type of

test recommended at the 1908 Congress that two sizes of bar be adopted, one 30 by 30 by 120 mm. with a span of 100 mm. having a nick 15 mm. deep with a base rounded to a 2 mm. radius; the other, one-third of the above dimensions. Small specimens are preferable to large ones, since the effect of segregation and defects are more pronouncedly revealed provided proper precautions are taken in locating the specimen.

The previously mentioned committee reported at the 1912 Congress of the I.A.T.M. that nicked test-pieces geometrically similar but differing in dimensions did not give the same energy of rupture per unit of volume. Consequently one must avoid comparing results gotten from nicked test-pieces differing either in shape or size unless the effect of the difference is known.

Tests by Fremont \* indicate that slight differences in the width of the saw-cut produce negligible effects on the results, while Thomas† experiments show that the angle between the sides of the nick may be varied from 0 to 30 degrees without materially influencing the energy of rupture. The experiments of Tetmajer, however, indicate that large variations in the curvature at the bottom of the nick or groove should be avoided. For specimens of the same dimensions, care must also be taken to make the depth of notch constant.

In tests on structural forms made of ductile material it is well to mark a number of equal intervals on both sides of the test-piece at sections where maximum stress will occur under impact. After impact, measurements of flow and compression will thus be made possible.

**125. Considerations Involved in the Selection of an Impact Testing Machine.**—The fundamental conditions which one must consider in selecting an impact machine have been briefly considered in Art 58. It must be pointed out, however, that machines are far from being standardized. In the drop machines, with the exception of the machine for testing rails (see Art. 60), there is a wide diversity of conditions. No standard relations of tup to anvil have been adopted, nor has there been any standardization of the height of drop. Fremont recommends a drop of at least 4 meters and an anvil weighing forty times as much as the tup. Experience seems to show that the hammer should be at least fifteen times as heavy as the specimen. When drop machines are used to apply blows smaller than required for rupture, some means should be provided to avoid secondary effects on the test-piece due to rebounding of the hammer.

A standard pendulum machine, Charpy type, has been adopted by the German Association for testing materials, but there are in use a large number of other types in some of which the anvil is much too light. In

\* *Proc. I.A.T.M.*, 6th Congress, 4.

† *Proc. A.S.T.M.*, Vol. 15, p. 63.

such machines the loss of energy through vibration must be great. Wide variation also exists in range of the velocity of impact. The cantilever method of supporting a specimen should be avoided, since it is very difficult to duplicate the end conditions in successive tests.

Friction losses are also worthy of attention, especially in drop machines. Friction effects in such machines may be made small by keeping the guides free from rust and wiping them with a cloth dipped in powdered graphite. Calibrations for friction losses can be made by measuring the elastic deflection of a spring attached to the anvil of the machine when subjected to different impacts and comparing the energy absorbed with that required to deflect the spring a like amount under static loading. The friction in drop machines equipped with a drum and tuning fork may be gotten by determining the slope of the velocity-time graph and comparing with the acceleration due to gravity.

The curvature of the knife-edges on the anvil and especially on the hammer is of importance. The hammer knife-edge on the Charpy machine has a radius of curvature of 2 mm.

**126. Testing.**—In drop machines where successive blows are to be struck, provision should be made to keep the specimen from jumping off of the supports during the test by means of slots or yokes near the ends; but fixing of the ends should be avoided. In tests on pendulum machines, the specimen must be placed tightly against the anvil. Where rupture is produced by a single blow it is best to adopt a uniform height of drop in order that the velocity of impact may be the same in all tests.

Pains should always be taken to see that every specimen is so placed that it receives the blow in the same position as its predecessors. Nicked specimens must be placed with the nick exactly opposite the hammer knife-edge. If such specimens are round, the base of the nick should be parallel to the hammer knife-edge when the latter is in contact with the test-piece. Timber specimens should be placed so that planes tangent to the annual rings will be perpendicular to the direction of the blow.

When the elastic limit is to be found, the deflection of the specimen is determined for successive blows in which the energy is progressively increased. The energy of the blow is then plotted against the square of the corresponding deflection. The elastic limit is the unit stress corresponding to the point at which the square of the deflection increases in faster ratio than the energy. In such tests it is impossible to measure the total energy absorbed by the specimen. The amount lies between the energy of the final blow and the sum of the energies in all the blows. In general, it is not possible to prescribe which of these quantities will form the better index for comparing ultimate resistance to impact, unless the form of the load-deflection curves are known. Consequently, a single

blow large enough to produce rupture is the preferable method of securing data on energy of rupture under impact.\*

**127. Observation after Rupture.**—In tests on metals, the shape of the fractured surface, its inclination with respect to axis of the piece, and its texture should all be recorded. If the metal is ductile the relation of the plane of failure with respect to the notch or nick should also be carefully observed. When a rolled piece is tested the limits of the scaled area are noteworthy. On polished specimens, the area showing lines of strain and the character of these lines should be noted. Microscopic examinations of the portion next to the break are often very useful in determining structural defects which cause peculiar results.

### HARDNESS TESTS

**128. Kinds of Hardness.**—Hardness as applied to metals, minerals and other solids is a term of variable meaning. Resistance to abrasion, to indentation, and to cutting have all been considered criteria of hardness; but no one of these serves in general, as a criterion for the others. Apparently abrasive resistance depends largely upon adhesion between the particles, resistance to indentation upon cohesion and cutting resistance upon both cohesion and adhesion. Therefore, it seems likely, and the results of tests † show, that for pure metals which are nearly homogeneous these different resistances are closely related. However for substances like cast iron, tempered steels, alloy steels, and alloys in which there is a decided difference in the mechanical properties of the constituent particles there appears to be no relation between these resistances. Since in practice distinct demands for the different sorts of hardness exist, it is quite desirable to standardize tests for the measurement of these properties. Thus far no single test has been devised which is in general well-adapted to measure all of these different kinds of hardness; nor is it likely that one ever will be devised. Relative hardness of similar substances may be gotten but no absolute standard appears.

**129. Types of Hardness Tests.**—The scratch test made with a diamond point is the oldest and simplest method of determining abrasive hardness. However, owing to difficulties in standardizing, this test has not come into general use. Probably the most satisfactory method of using it is that of Martens (see Art. 61).

For measuring cutting hardness, especially of cast iron, use has been made of the Bauer drill test. In this test the quantity of metal removed by a standard drill operating under constant speed and pressure for a certain time interval, is considered an index of cutting hardness.

\* For methods of testing axles and rails under impact consult the current *Standards of the A.S.T.M.*; also see *Engr. News*, Vol. 75, p. 701.

† See T. Turner's tests, *Jour. of Iron and Steel Institute*, 1909, No. 1, p. 426

Resistance to penetration has been experimented upon more scientifically than any other phase of hardness. The Rodman pyramidal punch which was attached to a falling weight was standardized by the French Commission and the following law deduced by Lieut. Col. Martel: \*

"For all forms of pyramids, for all weights of ram, and for all heights of fall, the volume of the displaced material of a given quality is equal to the energy of the blow ( $wh$ ) divided by a constant,  $D$ , which constant is the work or energy necessary to displace (by deformation) a unit-volume of that material. This constant is therefore characteristic of that material and may be taken as its index of hardness, or of its resistance to indentation."

At present two forms of test are being widely employed in this country, the Brinell ball method and the Shore scleroscope (see Art. 62 and 63). These methods will be considered in some detail.

**130. Objects of Indentation Tests on Metals.**—Indentation tests serve two very useful purposes: 1, to determine the quality or condition of parts which—on account of size or shape—cannot be subjected to other mechanical tests or which must not be destroyed in testing; 2, to determine hardness. In either case comparisons must be made with materials of like nature, since neither test furnishes a satisfactory indication of the comparative hardness or other mechanical properties of all substances. For example, with the scleroscope soft wood gives as high readings as hard steel, and india rubber gives readings equal to soft steel. Furthermore, comparisons between two dissimilar materials one of which had been tested by the scleroscope and the other by the Brinell method cannot in general be made. As an illustration, Devries showed that an alloy of 90 per cent copper and 10 per cent tin had a hardness greater than annealed tool steel when measured by the scleroscope but when measured by the Brinell method it was slightly harder than copper.

**131. Relations between Resistance to Indentation and Strength.**—Tests by several investigators, among which those of Abbott † and Turner ‡ are especially noteworthy, show that the ultimate tensile strength and the resistance to penetration of steels are linearly related. Abbott's results are based on about four thousand tests on five types of steel, each of which was heat treated in various ways. The upper limit of the strength for each type reported varied from 250,000 to 300,000 lb. Equations deduced by Abbott correlating strength and hardness appear in Table 2. The tests showed a better agreement between the results of the Brinell

\* The device was first used by Col. T. J. Rodman (U.S.A.) before 1860; see his *Report of Experiments on Metals for Cannon and Cannon-Powder*, 1861. For standardization, see *Commission des Méthodes d'Essai des Matériaux de Construction*, Vol. 3, p. 261.

† *Proc. A.S.T.M.*, Vol. 15, p. 43.

‡ *Jour. of Iron and Steel Institute*, 1909.

and scleroscope methods when the hardness readings were less than 300 (Brinell scale) than for higher values.

TABLE 2.—RELATIONS BETWEEN THE TENSILE STRENGTHS OF VARIOUS STEELS AND THEIR HARDNESSES (ABBOTT)

Kind of Steel.	Ultimate Strength in 1000 lb., in. <sup>2</sup> ( $S_t$ ) in Terms of		Relation of Brinell Hardness Number to Scleroscope Hardness Number.
	Brinell Hardness Number ( $B$ ).	Scleroscope Hardness Number ( $S$ ).	
Carbon steel.....	$S_t = 0.73B - 28$	$S_t = 4.4S - 28$	$B = 5.6S + 14$
Nickel steel.....	$S_t = 0.71B - 32$	$S_t = 3.5S - 6$	$B = 5.0S + 48$
Chrome-vanadium steel	$S_t = 0.71B - 29$	$S_t = 4.2S - 21$	$B = 5.5S + 27$
Low-chrome-nickel steel	$S_t = 0.68B - 22$	$S_t = 3.7S - 1$	$B = 5.4S + 33$
High-chrome-nickel steel.....	$S_t = 0.71B - 33$	$S_t = 3.7S - 3$	$B = 4.8S + 58$
All steels grouped together.....	$S_t = 0.70B - 26$	$S_t = 4.0S - 15$	$B = 5.5S + 28$

**132. Applications of Indentation Tests.**—Among the more important applications of indentation tests are the following: 1, to determine rapidly the carbon content in smelting iron and steel; 2, to determine strength in parts which cannot be subjected to regular strength tests; 3, to test the uniformity and degree of tempering or hardening in steels; 4 to test the effect of cold working of steels; 5, to ascertain the condition of finished parts without injury.

**133. A Comparison of the Brinell and Scleroscope Methods.**—In general, the Brinell test is less rapid, requires a larger surface for test, produces a more pronounced disfiguration of the part tested, penetrates to a greater depth and integrates the resistance of a larger number of particles than does the scleroscope. The chief error in the Brinell method arises in measuring the diameter of the impression. The error is due to the upward flow of metal around the sphere and the elastic deformation of the ball. This trouble is obviated by employing Devries' method of determining the depth of the impression Art. 62.

On the other hand, it appears that the scleroscope indications are affected by properties of the specimen other than hardness as the results tabulated below show. Furthermore, the readings of the scleroscope

Material.	Plate Glass.	Ivory.	Cork.	Hemlock	Hard Steel 0.79% C.	Soft Steel 0.14% C	Lead.
Scleroscope Reading	116	45	37	31	23	12	12

are influenced by the method of holding the specimen, by the smoothness of the surface, by the rigidity of the support on which the specimen



rests, by the shape of the hammer point, and also by the size of the indentation.

The scleroscope tests conditions existing less than 0.01 in. from the surface. It may, however, be applied to materials of any degree of hardness.

**134. Testing by the Brinell Method.**—The surface to be tested should be plane, free from scale, and smooth; it need not be polished. The American Soc. for Test. Mat. prescribes that the balls shall be made from steel containing from 1 to 1.2 per cent carbon and from 1 to 1.5 per cent chromium.\* These balls must be heat-treated to produce the maximum hardness consistent with the required crushing strength. The balls should measure within 0.0025 mm. (0.0001 in.) of 10 mm. (0.3937 in.). Balls should be remeasured after every test in which the hardness exceeds No. 600.

The standard load for the ferrous metals is 3000 kilograms (6614 lb.); for the minor metals and alloys, 500 kilograms is often used. Pressure should be applied slowly and kept on the specimen for thirty seconds. The distance between the center of the indentation and the edge of the surface should equal at least 2.5 times the diameter of the impression. The thickness of the specimen should be at least seven times the depth of the indentation.†

If the depth of penetration is measured by Devries' method, a microscopic reading to 0.001 mm. should be employed. If the diameter of the impression is desired, measurements should be made to at least 0.01 mm. on two mutually perpendicular diameters.

The general equation for Brinell's numbers is

$$H = \frac{P}{\frac{\pi D}{2}(D - \sqrt{D^2 - d^2})},$$

where  $H$  = hardness;  $P$  = pressure;  $D$  = diameter of ball; and  $d$  = diameter of impression.

In terms of the depth of impression,  $t$ , the equation is  $H = \frac{P}{\pi Dt}$ .

**135. Testing with the Scleroscope.**—The surface of the specimen should be horizontal. If the specimen is soft, a fine file is sufficient to smooth the surface; but if a hard steel is to be tested the surface should be ground on the side of a fine emery wheel. In tests on very hard metal, it may be advisable to polish the surface. In any event care should be

\* See *Methods for Brinell Hardness Tests of Metals*, E1-16, A.S.T.M. Standards, 1916.

† H. Moore, *Proc. I.A.T.M.*, 1909, 24.

taken to avoid injury through overheating. Mill scale, blisters or decarbonized products of annealing must be removed before testing.

When parts are standardized by this test it is necessary that each should be held in the same manner. Since the rebound of the hammer is largely affected by the rigidity of the test-piece, all small pieces should be securely held. A vise can be employed in most cases, but for pieces of peculiar shape a jig or plaster cast may be more suitable.

The apparatus must be held in a vertical position. For ease in reading, the light should be directed downward toward the scale so that the top of the hammer glistens. An estimation of the rebound of the hammer should be made before the test and the eye, held about 20 in. from the apparatus, should be sighted slightly below the calculated scale reading. For accurate reading of small differences in rebound, the magnifier may be set after the range has been obtained.

Always avoid testing the piece twice in the same spot. Surfaces composed of widely varying constituents should be tested at several points to secure the range in rebound.

## SHEAR TESTS

**136. Essential Conditions in Transverse Shear Tests.**—In order to obtain the true shearing strength of any substance it is necessary to develop in it, along a given plane, shearing stress only, unaccompanied by the bending stresses of tension and compression. To accomplish this it is necessary to concentrate the external forces of action and reaction on planes an infinitely small distance ( $dx$ ) apart. Any finite distance between these planes will develop a cross-bending action and its resultant direct stresses across the plane of shear. As it is impossible to so concentrate the external shearing forces, it is necessary to overcome the bending stresses due to non-concurrence of the external forces by preventing the bending of the specimen subjected to these forces. This is done by reinforcing the specimen between the shearing planes or by grooving the specimen in the planes of shear and supporting it by auxiliary clamps.\*

**137. Objects of Transverse Shear Tests.**—Although shear stress is present in nearly all cases where cross-bending exists, it becomes of a practical importance in only a limited number of instances. In the design of riveted joints, bridge pins crank-pins, short I-beams, and wooden beams especially, the shear stress must be considered. It is therefore of importance to be able to determine roughly at least, the shear strength of certain metals and of timber. It is however, impracticable in the transverse shear test

\* Both Dr. Kennedy and Mr. Barba grooved their specimens for double shear, and also held them in rigid forms. See Rep. French Commission, Vol. 3, Plate 19.

to determine the elastic limit or modulus of elasticity. These determinations on metals can be gotten best from torsion tests.

**138. Specimens for Shear Tests.**—For shear tests of metals, specimens of rectangular cross-section with a breadth equal to at least four times the depth are satisfactory for use with a Johnson shear tool. If round specimens are used, this shear tool should be modified so that the specimen is clamped in circular die.

Wooden specimens cut in accordance with Fig. 14, Ch. II, are satisfactory for use in the simple shear tool shown in the same figure. In preparing such specimens, one should remember that timber is weakest in longitudinal shear on planes tangent to the annual rings.

**139. Testing.**—To avoid bending, it is necessary that the clamps on the shear tool should grip the specimen tightly. Care must also be taken to place the specimen in the axis of the shear tool with the shear planes at right angles to the axis of the test-piece. To avoid eccentric loading of the specimen, it is necessary to apply pressure to the shear tool through a crossed knife-edge or through a spherical seat.

The speed of applying the load should not exceed 0.05 in. per minute for metals and not over 0.1 in. per minute for wood.

In addition to the ultimate load, observations of the shape of the fracture and its texture should be made.

### TORSION TESTS

**140. Objects.**—In general, torsional shear stress may be produced without bending, nevertheless one must not forget that on certain planes in a body under torsion these shear stresses combine to produce tensile or compressive stresses of equal intensity to the shear stresses. A torsion test differs from a transverse shear test in that the deformation acts over any predetermined length of the bar, and in that it varies from zero at the center of the bar to a maximum at some point on the outside. By using hollow cylindrical specimens having a large internal radius compared to thickness, it is possible to determine the ultimate shear strength of many materials. If, however, the tensile strength of the material is less than its shear strength, failure in tension will result from combined shear stresses.

For elastic materials, the elastic limit, yield-point, torsional modulus of rupture, modulus of elasticity and torsional resilience may be obtained, provided a cylindrical test-piece is used. Torsion tests are made not only on shafting but also on variously shaped members of machines or structures which are subjected to twisting couples. In the latter tests the twisting moment at the yield-point, the ultimate and the angle of twist at the yield-point and ultimate are the criteria of most use in design.

**141. Specimens for Torsion Tests.**—For tests on torsional properties of materials, the round bar (either hollow or solid) is the only satisfactory form of specimen. Since it is possible to multiply the twist of the specimen by using radial arms of considerable length, it is not important in tests on the modulus of elasticity that a long gage length be used. Ten inches is generally ample. If the test is to be carried to rupture, it is well to have the portions at the grips about 20 per cent larger in diameter than the gage portion. The transition, however, from the enlarged ends to the gage portion should be made by fillets of large radius.

When it is desirable to determine the torsional strength and ductility of structural elements having non-circular cross-sections, the test-piece should be geometrically similar in form to the shape under consideration. It should also be remembered that in such sections the maximum shearing stress is produced at points on the periphery nearest the axis. Formulæ for maximum intensity of stress in elliptical and rectangular sections are given in Art. 23.

**142. Testing.**—Descriptions of several machines and types of detrusion apparatus are given in Ch. II. The detrusion indicator should be accurate and sensitive to one one-hundredth of the estimated angle of twist in the gage length at the elastic limit. The machine should be so constructed that end tension is not exerted on the specimen. In testing wire, means for making the end tension constant should be provided. Care must be exerted in gripping to insure against slipping. If the specimen is hard and has cylindrical ends, it is well to cut fine closely spaced grooves along elements of the surface to afford a grip for the teeth of the jaws.

The cross-section of the specimen should be measured to one part in one thousand at several points along the gage length; and, to afford a rough determination of the angle of twist, a fine line may be scribed along a longitudinal element of the surface. If the modulus of the elasticity is to be determined, it is well to secure about ten readings of twist within the elastic limit and to cut down the increments about 80 per cent when the twisting moment becomes 90 per cent of the estimated moment at the elastic limit. The speed of the twisting head within the elastic limit ought not exceed 0.005 revolution per minute per inch of length of specimen. Beyond the elastic limit this speed may be greatly increased.

At the yield-point scaling will be observed on rolled sections. The location and the spreading of the scaling is of special note in tests on non-cylindrical shapes. At failure, the texture of the fracture and the inclination and shape of the fractured surface should be observed. As a measure of torsional ductility, the angle of twist per unit of length may also be determined.

## BEND TESTS OF METALS

**143. Significance of Bend Tests.**—A rough but very valuable test of the ductility of malleable metals is afforded by the various types of bend test commonly practiced in the shop. The test consists in sharply bending a bar or portion of a structural shape and noting the angle at which rupture occurs on the convex surface of the bend. If the material can be bent through an angle of  $180^\circ$  without rupture it is considered to have exhibited a high degree of ductility. The test is also used to

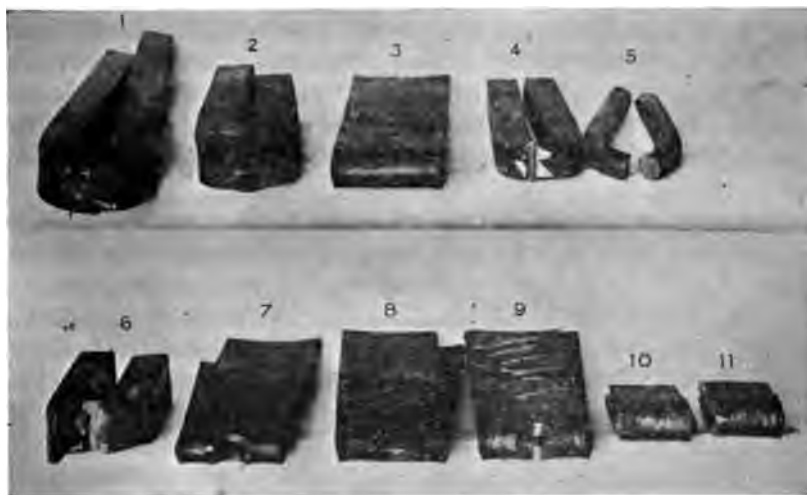


FIG. 16.—Results of Cold Bend Tests. (1)  $\frac{3}{4} \times 1$ -in. Wrought Iron Bar, Note Cracks; (2)  $\frac{3}{4} \times 1$ -in. Mild Steel Bar; (3)  $\frac{3}{4} \times 2$ -in. Mild Steel Flat; (4)  $\frac{3}{4} \times \frac{3}{4}$ -in. Cold Rolled Bar; (5)  $\frac{3}{4}$ -in. 0.90% Carbon Steel Rod; (6)  $\frac{3}{4} \times 1$ -in. Wrought-Iron Bar Nicked and Bent, Note Cracks at Left; (7) Mild Steel Flat Punched and Bent with Die Side Outside; Note Cracks; (8) Mild Steel Flat Punched and Bent with Die Side Inside; (9) Mild Steel Flat Drilled and Bent; (10) Mild Steel Flat Sheared along Edges and Bent with Die Side Inside; (11) Mild Steel Flat Sheared along Edges and Bent with Die Side Outside, Note Cracks.

ascertain the effects of certain shop operations punching, drilling, and shearing—on ductility. Fig. 16 shows results bend tests on wrought iron and steel specimens. This test is a more severe measure of ductility than the tension test but has not been so carefully standardized. The greatest advantage of the test is that it may be made without expensive equipment, consequently methods of standardizing should not remove it beyond the range of ordinary shop appliances.

**144. Various Kinds of Bend Tests.**—Cold-bend, quench-bend, hot-bend and nick-bend tests are all used more or less. Of these the cold-bend test is most commonly practiced. This test is ordinarily made at normal shop

temperatures; sometimes to test ductility at low temperatures, specimens are artificially cooled. Cold-bend tests are made on structural steel for bridges, buildings, ships, locomotives, reinforced concrete, rivets, splice bars and on the various grades of wrought iron. The test, as commonly made, measures ductility of the metal at normal temperatures only.

For determining ductility of metal parts subjected to alternations of high and low temperatures, use is sometimes made of the quench-bend test. The specimen is first heated to a yellow heat and suddenly cooled by plunging in water at a temperature of about 80° F. It is then subjected to the bend test. Boiler, fire-box, boiler-rivet steel, and stay-bolt iron are often subjected to this test.

To measure the ductility of wrought iron at welding heat, a hot-bend test is sometimes specified. The specimen is heated to a temperature of 1700 or 1800° F. and immediately bent. This test is useful in detecting a high-sulphur content.

Nick-bend tests are made on wrought iron to detect the presence of a crystalline structure. The A.S.T.M.\* requirement for the nick-bend test of engine-bolt iron follows: "The test specimen, when nicked 25 per cent around with a tool having a 60-degree cutting edge, to a depth of not less than 8 nor more than 16 per cent of the diameter of the specimen, and broken, shall show a wholly fibrous fracture."

**145. Specimens for Bend Tests.**—The cross-section of specimens for bend tests may be round, square or rectangular; finished shapes are sometimes tested. Rods and bolt stock are generally tested without machining. When specimens are sheared from plates the edges should be planed and the corners rounded with a file. If this is not done, cracks may start at the corners due to weakening of the metal by the shearing process. Martens advocated that the length of flats should be eighteen times the thickness, the breadth three times the thickness, and the radii of the sheared corners should be one-fourth of the thickness.†

If the specimen is nicked it is preferable to use a planer or milling machine rather than the cold chisel, although the latter method is the easiest. Martens recommended that the depth of the groove be between 10 and 20 per cent of the thickness of the test-piece. It should extend across the surface which is under tension during the test.

For bend tests on perforated specimens the hole should be located in the center of the test-piece. It should be punched before the specimen is sheared from the plate. The diameter of the hole should bear a fixed relation to the thickness of the test-piece. Martens recommended that the diameter of the hole should be twice the thickness and the width of the test-piece should be five times its thickness.

\* See A.S.T.M. Standards, 1916, Standard Specifications for Engine-bolt Iron.

† *Handbook of Testing Materials*, p. 319.

**146. Various Methods of Testing.**—Methods of starting and closing the bend vary considerably, and herein is the one of the main causes for discrepancies in this test. On small specimens it is possible to use a blacksmith's sledge and anvil. The specimen is first partially bent over the edge of the anvil. It is then held in a vertical position by tongs and the bend closed by blows from the sledge. A steam hammer is very commonly used for this test, the bend being started in a V-block. Owing to the difference in the masses of hammers and the variation in number and energy of blows, these methods will give variable results. If they are to be used, the number of blows should be stipulated; and if sledge is permitted, its weight should be specified. Small specimens may be bent in a strong vise. When this method is adopted, the free end of the test-piece should be shackled to a long lever to prevent bending beyond the curved portion. After bending through  $90^\circ$  the other end may be gripped in a similar shackle and the test completed.

Bending by machines, although less severe than the hammer method, yields more concordant results. Descriptions of certain devices for this purpose may be found in Art. 54 and 55.

When nicked or perforated specimens are tested, the test-piece should be so placed that the bend will occur at the minimum cross-section. In testing punched specimens the die side should be subjected to tension in the bend test.

**147. Influence of Thickness of Specimen.**—If we consider that the neutral plane remains in the center of the test-piece during bending, the per cent elongation of the outer fiber is  $100\frac{t}{2R}$ , where  $t$  is the thickness and  $R$  the radius of curvature of the neutral surface. Tests show that although the above assumption is erroneous, the elongation of the outer fiber does vary directly with the thickness and inversely with the radius of curvature. Consequently, it is customary in specifications to increase the radius of curvature for thick test-pieces. Thin steel plates are commonly bent flat through  $180^\circ$  (roughly  $R=t$ ); those above  $\frac{3}{4}$  in. are bent about a pin having a diameter equal to the thickness of the plate. It is common practice to bend rods about pins of equal diameter. (For examples see specifications for various classes of steel and wrought iron in the current *A.S.T.M. Standards*.)

**148. Observations during Tests.**—Owing to the congestion of metal on the concave side of the bend and restricted lateral expansion, the neutral axis of the specimen must approach nearer to the concave side as the angle of bend increases. Also, more of the specimen is severely stressed and deformed as the bend angle increases. Consequently the angle through which the specimen is bent without cracking forms a crude index of ductility. It is good practice in tests for comparative purposes to observe

both the angle at first crack and at rupture. To make such measurements with accuracy the use of a cylindrical plug gage or a cone graduated on one element will be found convenient. Most American specifications are so worded that no observations are required until the specimen has been bent a prescribed maximum amount.

### DRIFTING TESTS OF METALS

**149. Their Character and Significance.**—Drifting tests like the bend tests, are such as may be applied in the workshop and by the workmen themselves with their ordinary shop appliances. The test consists in punching or boring holes of given diameters (varied with the thickness of the plate) at given distances from the edge of the plate or structural form, and then enlarging it by driving in it a drift-pin, as shown in Fig. 17, the percentage of enlargement without cracking being a very good indication of the ductility of the metal. To serve as a criterion of comparison, however, it must be reduced to fixed rules, the same as all other kinds of tests.

A specification commonly used in France is as follows:\*

Wrought-iron bars shall be cut both with and across the grain, 3 in. wide, and three holes punched,  $\frac{3}{8}$  in. in diameter and  $2\frac{1}{2}$  in. apart, along the central line of the plate. These holes shall then be enlarged, beginning with the central one, and using a drift-pin which increases its diameter at the rate of 1 in 10. Plates 0.20 in. thick should submit to an enlargement of the  $\frac{3}{8}$ -in. hole to a diameter of 1 in.; plates 0.25 in. thick should enlarge to 1.2 in. diameter; plates 0.30 in. thick should enlarge to 1.32 in. diameter; and plates thicker than 0.32 in. should enlarge to from 1 in. to 1.3 in., according to quality, without showing any sign of failure.

Steel plates, similarly prepared, of 57,000 lb./in.<sup>2</sup> tensile strength should enlarge to 1.6 in. diameter, after annealing and to 1.5 in. diameter after hardening in water. Steel plates of 57,000 to 64,000 lb./in.<sup>2</sup> tensile strength should allow a  $\frac{3}{8}$ -in. hole to enlarge to 1.5 in. diameter after annealing and to 1.4 in. diameter after hardening in water.

### THE VALUE OF MECHANICAL TESTS

**150. A Résumé of the Utility of the Principal Mechanical Tests.**—In the following paragraphs will appear a brief summary of the prop-

\* That of the Eastern Railway Company.



FIG. 17.—Drifting Test on  $\frac{1}{2}$ -in. Steel Angle. A  $\frac{1}{2}$ -in. Hole Drifted to  $2\frac{1}{8}$  in. in Diameter. (*Engr. News*, Vol. 33, p. 272.)



erties revealed by the various mechanical tests and the principal uses now made of these tests.

*Tension Tests.*—With ductile materials the strength, ductility and toughness are revealed. With brittle materials, strength only is measured. If an extensometer is used the modulus of elasticity may be found.

Tensile tests as a class are the most valuable of the mechanical tests especially for ductile metals, fabrics, paper, cloth, yarn and rubber. They are of less value in testing brittle materials, although they are much used for tests of cementing materials.

*Compression Tests.*—With ductile materials the yield point and a poor measure of toughness can be determined. Ordinarily with brittle materials strength only is gotten. If a compressometer is used the modulus of elasticity may be obtained.

Compression tests are of great value in determining the strength of brittle materials like wood, concrete, cast iron, brick and stone.

*Transverse Tests.*—With ductile materials the yield point and modulus of elasticity may be found. With brittle materials a criterion of strength (modulus of rupture), the flexibility, and the toughness may be determined.

This is an inexpensive and very valuable test for brittle materials such as cast iron, wood, brick, cement and concrete.

*Bend Tests.*—Cold, hot, quench and nick-bend tests are sometimes used. The first three of these tests are measures of ductility for metals subjected to the respective conditions. Cold-bend tests detect very high carbon contents in steels normally cooled; they may detect brittleness due to a high phosphorus content or improper treatment in working or rolling. Failure in the hot-bend test may be due to a high sulphur content. Failure in the quench-bend test is likely to result if the carbon content is above 0.20 per cent. The nick-bend test is useful in determining the structure of the metal and to detect imperfect methods of manufacture.

These tests afford a simple, inexpensive and valuable means for the shopman to determine the suitability of ductile metal like boiler plate, fire-box steel, reinforcing bars, rivet metal, engine-bolt iron and stay-bolt iron.

*Hardness Tests.*—The resistance of metals and other materials to indentation is the form of hardness most often tested. The Brinell ball apparatus and the Shore scleroscope are the devices most used for the determination of this form of hardness. The Brinell test permanently deforms the specimen but is a better index of hardness (within the limits of the hardness of the ball) than the scleroscope test. The scleroscope indications are influenced by other factors than resistance to indentation.

but the apparatus may be used on harder material than the Brinell device and does not seriously alter the surface of the specimen.

The indentation tests afford an inexpensive and very valuable means for determining the quality of hardened and tempered metals, for standardizing the hardness of parts, and for detecting flaws in parts which on account of peculiarity in form can not be subjected to other mechanical tests.

*Impact Tests.*—These tests afford a valuable measure of shock resistance for brittle materials. If the shock resistance of ductile metals is desired, a nicked specimen is generally necessary.

The test is most used for cast iron and wood, and for structural parts like rails, axles, shafting and forgings.

*Shear Tests.*—The transverse shear test affords an imperfect measure, due to the existence of bending stresses, of shearing strength.

These tests are used somewhat for metals, wood and for riveted and nailed joints.

*Torsion Tests.*—A measure of strength (the computed twisting strength), ductility, and toughness in torsion is afforded by this test. The shearing modulus of elasticity may be gotten if a proper torsion indicator is used.

The test is occasionally used on ductile metals. shafts and parts which are to be subjected to twisting.

## CHAPTER IV

### CHARACTERISTICS, PHYSICAL PROPERTIES AND USES OF WOOD

**151. Importance of Wood.\***—According to Government estimates the original stand of timber in the United States was approximately 5,200,000,000 board feet. During the past century, however, the forests of the New England States, the Allegheny Mountain region, and the Great Lakes States have been considerably depleted, and large inroads have been made upon the yellow pine forests of the Southeastern States. As a result of rapid cutting, forest fires, and waste only enough trees remain in our forests to produce 2,800,000,000 board feet of lumber; and nearly half of this amount lies in Oregon, Washington, Idaho and California. During the past decade the rate of consumption of timber from all causes, both useful and destructive, has amounted to nearly 100,000,000,000 board feet per annum, the remainder being wasted or destroyed by fire. Inasmuch as the rate of growth is perhaps one-third of the yearly consumption, it is evident that our forests are being impoverished at the rate of more than 60,000,000,000 board feet each year.

If cutting were to continue at the present rate, it would be only a question of a half century when our supply of timber would be gone. This is not likely to happen for two reasons: The price of timber is rapidly increasing, thus forcing the use of substitutes where possible; and there will undoubtedly result from the increase in value of the timber more conservative management of forests and less waste in the utilization of wood. Substitutes for wood in bridge building and car construction and for fence posts are already being used to a considerable extent. It is also likely that the use of substitutes for wooden poles, piles and shingles will be more generally employed in the future. Nevertheless, there will still be a great demand for wood not only in these forms of construction, but for ties, finishing, furniture, cabinets, implements, cooperage, and many minor uses in which no good substitute for wood has been found. However, since slightly less than one-fourth of the standing timber is held under government ownership, it will be necessary for private corporations as well as the government, to practice far better management of forests than has been common in the past if future generations are to be given their just heritage of timber.

\* Figures in this article are taken from a paper on *Structural Timber in the United States*, by H. S. Betts and W. B. Greeley, *Proc. Int. Engr. Congress, San Francisco* 1915.

Engineers are concerned principally with uses of timber for structural purposes. Each year they direct the use of pine, hemlock, Douglas fir, cypress, oak, redwood, chestnut, ash, spruce, and cedar aggregating a value of perhaps an hundred million dollars. Consequently it is quite necessary that they should be well informed concerning the properties and uses of this very valuable building material, in order that they, too, may use it to best advantage and conserve it wherever possible.

#### GENERAL CHARACTERISTICS OF WOOD \*

**152. Structure and Appearance.**—Although color, weight, smell, and resonance are often helpful, the structure of wood is the only reliable means of identifying species. Structure is closely related to all the mechanical properties of wood and very often furnishes an explanation for observed differences in these properties. Furthermore, structure and color determine the beauty of wood, as seen in the hard pine ceiling, the quartered-oak desk, the "bird's-eye" maple dresser, the mahogany paneling, and many other examples of decorative woodwork. For the engineer a knowledge of structure is of most importance because of the relation of structure to species and mechanical properties. We shall, therefore, briefly consider the different classes of timber and certain of their more prominent structural characteristics before taking up their properties.

**153. Classes of Trees.**—Two classes of trees furnish practically all of the structural timber of the United States. They are the gymnosperms (naked seed-leaved trees), of which the conifers are the important family; and the dicotyledons (trees having two seed leaves), which are commonly termed broad-leaved trees. A third class, the monocotyledons, of which the yuccas and palms are the more prominent native members, are used to a small extent in some of the Southern States.

Sometimes trees are also classified by the way that they shed their foliage. The conifers which are of most importance in the lumber industry are evergreen, although the larch and bald cypress shed their needles annually. On the other hand nearly all of the broad-leaved trees are deciduous in our northern latitudes.

Frequently, in the trade, the lumber of broad-leaved trees is called hardwood, and that of the conifers softwood. Here again the terms are inexact; since poplar, basswood, and horse-chestnut (broad-leaved trees), are soft woods, while longleaf pine and yew (conifers), are hardwoods.

The classes of conifers which are of most importance structurally are pine, fir, hemlock, cypress, spruce, redwood and cedar. Oak, maple, red

\* In the preparation of the remainder of this chapter frequent reference was made to *Bulletin No. 10 of the U. S. Forestry Div. by F. Roth, to Economic Woods of the United States*, and *Mechanical Properties of Wood*, by S. J. Record.

gum, poplar, chestnut, birch, beech, basswood, elm, ash and hickory are much-used broad-leaved trees.

Both conifers and broad-leaved trees grow in diameter as well as in height through the addition of yearly layers of wood, each of which forms immediately under the bark. On the other hand, the yuccas and palms increase principally in height. Although alike in manner of growth, conifers and broad-leaved trees differ markedly in structural detail and in character of wood elements. The wood of the conifers is characterized by a marked likeness in the wood elements and in their arrangement, while the wood of the broad-leaved trees consists of a greater variety of fibers and cells which lack the regular arrangement seen in the conifers.

**154. Structure of Wood in General.**—If one examines a sawn log of well-grown structural timber, he will perceive a small pith at the center of the cross-section surrounded by numerous concentric rings of wood which is, in turn, encircled by the bark. The concentric rings represent the layers of wood added each year during the life of tree. These *annual rings*, therefore, furnish valuable information regarding the age of the log, the rapidity and the uniformity of its growth. The thickness of the annual rings will be found to vary greatly in different trees of the same species and in different parts of the same cross-section. Trees grown in the open or after a forest has been cleared (second growth) exhibit wider rings than those which grow more slowly in the forest. Generally the rings are widest at the center and become narrower nearer the bark. Also the width of the same ring will vary from the bottom to the top of the tree. It is widest at the bottom in young thrifty trees, but in the old trees of the forest it is widest near the top. In thrifty forest trees the center rings are often a half-inch or more in width, but in stunted specimens rings less than one-two-hundredths of an inch wide are found. With the conifers a medium rate of growth is conducive to high strength and toughness; whereas a rapid growth is desirable to produce maximum toughness and strength in hardwoods like hickory.

Closer observance of a single annual ring discloses that it is not uniform in composition. In many of the conifers the interior of the ring is decidedly lighter in color than the outer portion; the exact opposite is true in the case of the oaks, while in a cross-section of hard maple no great variation in the color of a single ring is visible to the naked eye. These differences in color are due largely to variations in the size and structure of the wood cells and fibers. In the *non-porous* conifers the color of the earlier wood is due principally to cells which are less compact than those grown more slowly in the summer (Fig. 1). This difference in color and ring structure is not, however, very pronounced in white pine. The *ring-porous* woods like the oaks and the hickories owe the darker color of their early wood to the presence of numerous pores, each of which is large enough

to admit a pin point, and which are almost absent in the later wood (Fig. 2). *Diffuse-porous* woods, like beech and maple (Fig. 3), show little variation in the appearance of the early and late wood because of the more uniform dispersion of the pores across the ring. Owing to the time of

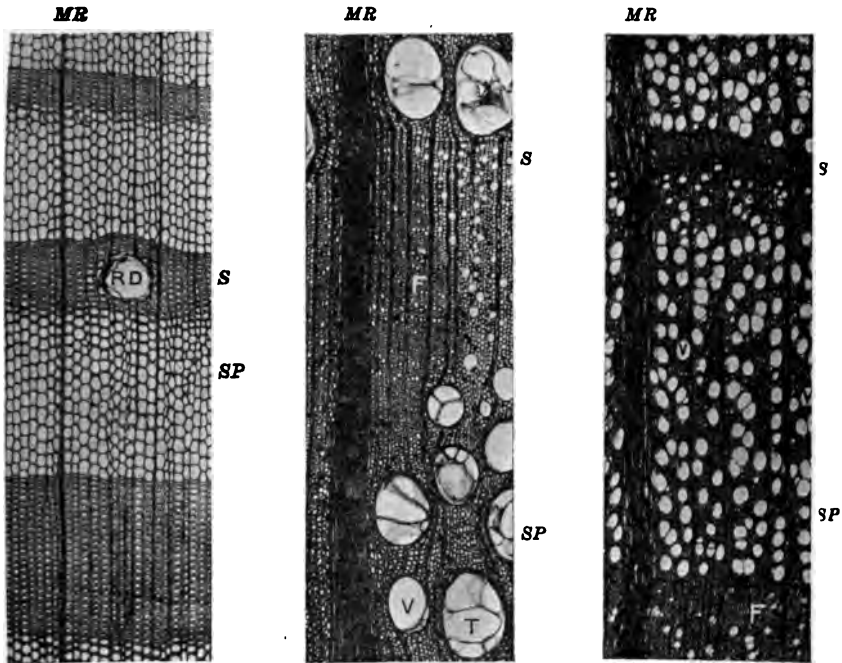


FIG. 1.

FIG. 2.

FIG. 3.

FIG. 1.—Cross-section of Longleaf Pine, a Nonporous Wood. (Note absence of pores; resin duct, RD; thick cell walls in summer wood, S; thin walls in spring wood, SP; narrow medullary ray, MR.)

FIG. 2.—Cross-section of White Oak, a Ring Porous Wood. (Note large vessels, V, in spring wood, SP, some being clogged with tyloses, T; broad medullary ray, MR; also fibers, F.)

FIG. 3.—Cross-section of Beech, a Diffuse Porous Wood. (Note the uniform dispersion of vessels, V; fibers, F; medullary ray, MR.)

[Figs. 1, 2, and 3 are magnified about 27 diameters. Reduced from photomicrographs prepared by the Forest Products Laboratory, Forest Service, U. S. Dept. of Agric.]

growth in our climate, the early wood is called *spring wood*, and the later wood is termed *summer wood*.

Since the summer wood is denser, harder and stronger than the spring wood, it follows that the percentage of summer wood in a stick of timber affords information concerning the mechanical properties of it. (See Art. 238.) Furthermore, the contrast between the spring and summer woods is sometimes of assistance in distinguishing different timbers of the same class—the pines for example.

The characteristic appearance of the annual rings after the log has been sawn into boards is seen in Figs. 4 and 5. Two important differences will at once be noted in these figures. The more open spring wood of the pine board is lighter colored than the denser summer wood, whereas in the oak the porous spring wood makes it appear darker than the more compact summer wood. Also, on the transverse section of the oak there are well-marked radial bands or *rays* which appear as light-colored parallel lines on the radial section. These rays, often termed *medullary rays*, are quite pronounced in many of the hardwoods but are less easily observed

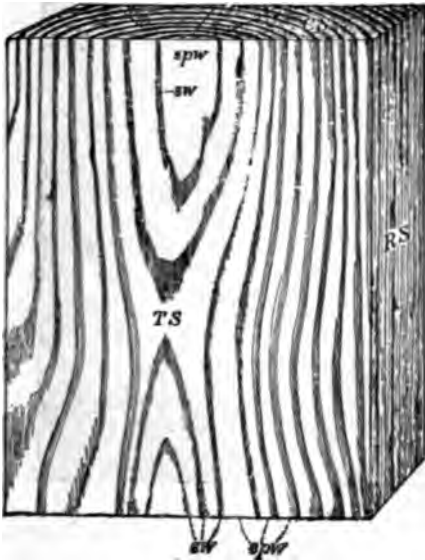


FIG. 4.—Board of Pine. CS, cross-section; RS, radial section; TS, tangential section; sw, summer wood; spw, spring wood.

by the eye in the softwoods. They are, however, present in both classes of trees and serve as distributors of water and food supplies between the bark and adjacent layers of wood. In the oaks the rays lend the beauty to the grain of the quarter-sawn lumber. In all classes of timber they greatly influence shrinkage and thereby affect mechanical properties, as we shall see later.

As a tree grows the cells near the pith gradually become inactive so far as the life of the tree is concerned. They do not decay, however, but remain a firm and strong support to the tree. This inner lifeless portion of the trunk is called the *heartwood*. The *sapwood* is the surrounding envelope, between the heartwood and the

bark, which carries the water and alternately stores and supplies the food for the growing portions of the tree. Generally, through the infiltration of pigments, the heartwood is darker in color than the sapwood, although the contrast is not always marked. Red cedar, redwood, yew, tamarack, elm and birch exhibit a strong contrast in the color of the sapwood and heartwood; but in fir, hemlock and spruce the contrast is largely wanting.

The proportion of sapwood varies considerably in various parts of the same tree. Generally the envelope of sapwood becomes thinner near the top of the tree and in the limbs. Very thrifty rapid-growing trees generally have a larger proportion of sapwood than trees of like species having a stunted growth, but the latter often have more rings in the sapwood.

The width of sapwood varies considerably for different kinds of wood; it is small for longleaf pine, white pine and chestnut, and great for loblolly pine, Norway pine, ash, maple, hickory and gum. Occupying the peripheral portion of the log the sapwood always forms a large proportion of its mass.

In old tree trunks the sapwood is likely to be weaker, freer from knots, and more susceptible to decay than the heartwood. In comparatively young timber the difference in strength between heartwood and sapwood is small. Sapwood is more readily impregnated with preservatives than heartwood. Some claim that the sapwood of hickory is stronger and tougher than the heartwood, but tests by the Forest Service \* have failed to substantiate this prejudice.

#### 155. The Grain of Wood.—

The comparative width of annual rings, the direction and the arrangement of the cells and fibers are the causes of the *grain* of the wood. Thus trees of rapid growth having wide annual rings produce coarse-grained wood, while those of slower growth produce wood with narrow rings or fine grain. When the wood elements are straight and run parallel to the pith the wood is said to be straight-grained. Often, however, the elements are twisted around the axis of the tree causing spiral grain. Frequently in cypress and gum several rings of fibers will run oblique to the axis of the tree in one direction, whereas the next layers of elements are oblique in the opposite direction; such wood is cross-grained. Wavy grain is caused by large undulations in the wood elements, generally on radial surfaces, curly grain by small undulations; oak, ash, cherry, birch, and maple often exhibit curly or wavy grain.

Since straightness of grain is of great importance in wooden beams and tension members, careful examination should be made to eliminate cross-

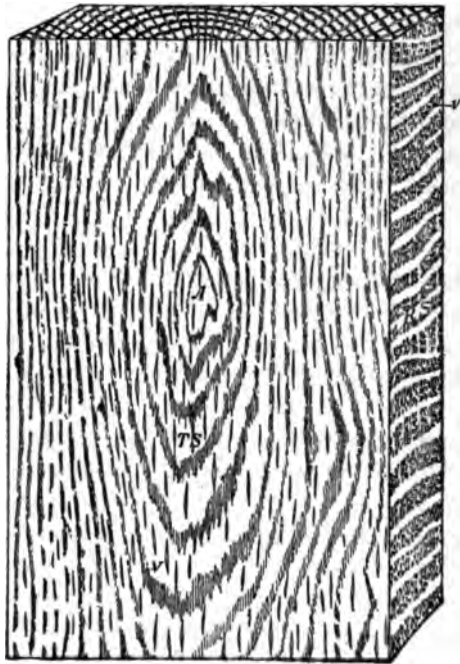


FIG. 5.—Board of Oak. CS, cross-section; RS, radial section; TS, tangential section; *v*, vessels or pores, cut through; *A*, slight curve in log which appears in section as an islet.

\* *Bulletin* No. 80, p. 50; also see Art. 239.



grained or spiral-grained timbers. The splitting radially of a small stick is about the only safe test for detecting spiral grain; although, if season checks are present, their course along the timber forms a good tell-tale. As a general rule split timber is stronger than that which is sawn.

**156. Defects in Timber.**—Besides irregularities in the character of the grain, there are, from a structural standpoint, three important classes of defects in timber—knots, checks and shakes.

*Knots* are the beginnings of branches which have been surrounded by the parent stem. Generally the piths of branch and stem join and the annual rings of the branch are continuous with the lower rings in the stem. The fibers above the branch, however, do not continue into it. This structural arrangement accounts for the fact that a cleft started below the knot will generally run into it, but if started above the knot, this does not happen. When the branch dies the stub is gradually covered by subsequent layers of fibers. The portion of the branch encased while living will make a *sound knot*, while that encased after the limb died will make either a *loose knot* or *decayed knot*. Sound knots are generally much harder than the surrounding wood. Knots greatly affect the workability, cleavability, shrinkage, and the strength of wood. (Art. 240.) Knots in the coniferous boards are likely to be filled with resin, a condition which renders them difficult to cover with paint.

*Checks* are radial cracks produced by unequal stresses set up in the lumber during the seasoning process. (See Art. 161.)

*Ring shakes* are separations between adjacent annual rings. They are thought to be due to bending of the tree by the wind. Often they are invisible in green timber but become apparent after seasoning.

*Heart shakes* are radial cracks emanating from the pith in the trunks of very old trees. It is likely that shrinkage of the heartwood while the tree is still standing produces these defects.

Both shakes and checks adversely affect the durability of timber because they readily admit moisture and air. If present near the neutral plane of a beam they may materially weaken its resistance to horizontal shear.

**157. Color and Odor.**—The colors exhibited by different woods or by the sapwood and heartwood of the same tree are due to chemical compounds impregnating the wood substance. Freshly formed wood is nearly colorless, while the sapwood is lighter than the heartwood in all species having any pronounced demarkation between the two. Exposure to air deepens the color of wood and immersion under water darkens it very materially. When wood is attacked by fungi it loses its luster and becomes dull in appearance.

Color, together with structure, is an important determinator of the beauty of mahogany, cherry, black walnut, red gum and sycamore. In

many woods the color of the heartwood is a valuable aid to identification. Thus the white of the white spruce, the yellow of the osage orange, the yellow-brown of the tamarack, the light brown of the chestnut, white oak and sycamore, the red-brown of the red oak, red cedar and red gum, and the dark brown of the black walnut are all distinctive marks.

Odor is also due to chemical compounds in the wood substance. It is much reduced by exposure to the weather but can generally be determined by making a fresh incision in the wood. Heartwood gives a stronger odor than sapwood. Odor is undesirable in woods which are to be used as food containers. In cedar chests, however, it forms a protective against moths. As a means of identification the resinous odor of longleaf pine, the aromatic odor of cedar, the sour smell of oaks, the smell of kerosene emitted by catalpa are noteworthy. Decayed timber often has a smell which is pronouncedly different from the odor of the sound wood; decaying red oak for example smells like heliotrope.

#### PHYSICAL PROPERTIES OF WOOD

**158. Density and Specific Weight.**—Experiments have shown that the specific gravity of all wood substance is approximately 1.55; whereas the density or apparent specific gravity (the ratio of the weight of a wooden block to the weight of an equal volume of water), varies from 0.11 in the case of balsa wood to 1.29 for black ironwood. This discrepancy between the specific gravity and the density of wood is due to the porosity of the wood structure. It underlies our notions of light wood and heavy wood and accounts for the buoyancy of wood in water.

The percentage of moisture in the wood has a very large effect upon the specific weight. Thus green wood is heavier than dry wood, the green sapwood of a given log is generally heavier than the accompanying heartwood, and the wood of the green sapling is heavier than that from the old tree. Since the moisture content greatly influences the specific weight, true comparisons of this important property can only be made on dry specimens.

Considering dry wood, we may say that the wood of a given tree is heaviest in the lower portion of the trunk, lighter in the limbs and branches and lightest in the roots. The heaviest timber in old pine is found about half way between the center and the bark where a medium rate of growth has obtained. In young pines and in ring-porous woods like oak the heaviest wood is at the center of the stump. In general the dry heartwood is much heavier than the dry sapwood. The dry wood of the oak sapling is heavier than that of the old tree, but in the pines and spruces the reverse is true. In the conifers high density is favored by a medium or slow rate of growth and a high percentage of summer wood. Thus the

specific weight of shortleaf pine, redwood, western hemlock and Douglas fir has been found to vary linearly with the per cent of summer wood.\* In the ring-porous woods like oak, chestnut, and elm a rapid rate of growth is attended by the production of a large proportion of thick-walled fibers, thus causing a high specific gravity; but in the diffuse porous woods like maple, birch and beech, where the cell structure within the rings is more homogeneous, no relation between rate of growth and density is apparent. As in coniferous timber, however, so in wood from broad-leaved trees, the density increases with the proportion of summer wood.

Since the specific weight of dry wood is often a very important criterion of the strength and other mechanical properties of timber (see Art. 236), it is well worth determining. In the laboratory small discs about 1 in. thick are cut from different portions of large sticks so that the variation in weight and moisture content may be represented. The pieces are then carefully weighed, dried in an oven to constant weight at 100° C., and again weighed. The specific weight is then computed from the volume and dry weight of the disc. Density is frequently based upon the volume at test and the oven-dry weight. In the field a rough but satisfactory method of determining density is afforded by boring holes of known diameter and depth in different portions of a timber and determining the specific weight from the dry weight of the shavings and the volume of the hole.

The range in average values for the specific gravity and specific weight of the more important native timbers are given in the table opposite. Other results are given in Table 1, Ch. VI. It will be understood that individual specimens may vary considerably from the values tabulated.

"Since ordinary lumber contains knots and also more water than is here assumed, and also since its dimensions either exceed or fall short of perfect measurement, the figures in the table are only approximate.

"Thus 1000 feet, B. M., of long-leaf pine weighs:

	Pounds.
" Rough and green . . . . .	4500
Boards rough and seasoned . . . . .	3500
Boards dressed and seasoned . . . . .	3000
Flooring, matched dressed and seasoned . . . . .	2500
Weather-boarding, beveled and dressed . . . . .	1500"

**159. Moisture in Wood.**—Water is found in three portions of wood: (1) it constitutes over 90 per cent of the protoplasm in the living cells; (2) it saturates the cell walls; and (3) it fills, more or less completely, the pores of the lifeless cells. It occurs in all three portions of the sapwood but only saturates the cell walls in the heartwood. In the sapwood of

\* See Fig. 17, *Bull.* No. 108, Forest Service.

white pine, containing 100 per cent of water (in terms of the dry weight of the wood) about 5 per cent of the water is found in the living cells, 35 per cent in the cell walls and the remaining 60 per cent in the empty cells.\* The water in the cells of the sapwood is drawn upward into the leaves of the tree and converted into sap. The sap is conducted down through the bark and provides nourishment for the wood forming immediately beneath the bark (the *cambium*). The water in the sapwood contains slight traces of mineral salts and during the winter it may carry traces of sugar, organic acid and gum. These substances apparently exercise little effect upon mechanical properties of wood, however, since only a trace of them can be found in it.

WEIGHT OF KILN-DRIED WOOD OF DIFFERENT SPECIES<sup>1</sup>

Common Name of Species.	APPROXIMATE.		
	Specific Gravity.	Weight of—	
		1 cubic foot.	1000 feet of lumber.
(a) Very heavy woods: Hickory, oak, persimmon, osage orange, black locust, hackberry, blue beech, best of elm, and ash.....	0.70-0.80	42-48	3700
(b) Heavy woods: Ash, elm, cherry, birch, maple, beech, walnut, sour gum, coffee-tree, honey-locust, best of Southern pine, and tamarack.....	.60- .70	36-42	3200
(c) Woods of medium weight: Southern pine, pitch-pine, tamarack, Douglas spruce, Western hemlock, sweet gum, soft maple, sycamore, sassafras, mulberry, light grades of birch and cherry.....	.50- .60	30-36	2700
(d) Light woods: Norway and bull pine, red cedar, cypress, hemlock, the heavier spruce and fir, redwood, basswood, chestnut, butternut, tulip, catalpa, buckeye, heavier grades of poplar.....	.40- .50	24-30	2200
(e) Very light woods: White pine, spruce, fir, white cedar, poplar....	.30- .40	18-24	1800

<sup>1</sup> Quoted from Roth, *Bulletin* No. 10, Forest Service.

The wood immediately under the bark carries most of the water, and the proportion in any one cross-section varies as the percentage of sapwood. It is, therefore, greatest in the roots and greater in the limbs and branches than in the trunk. The wood of saplings contains more moisture

\* Roth in *Bull.* No. 10.

than that of old trees. Also, in the timber of thrifty trees, the per cent moisture is greater than in those of stunted growth.

It is impossible to remove moisture entirely from wood without setting it on fire, but all excepting a couple of per cent may be eliminated by drying at 100° C. Moisture determinations may be made on the same specimens used for determining density. The per cent moisture should, however, always be referred to the dry weight (100° C.), in order to have a constant basis for comparison. The moisture content of green wood for a number of species will be found in Table 1, Ch. VI.

**160. The Drying of Timber.**—It is just as essential to dry properly, or “season” wood, as it is to condition concrete by proper storage, or steel by proper heat treatment. Nevertheless the proper conditioning of timber has received far less consideration than it deserves. The chief reasons for drying timber are as follows: (1) to decrease shrinkage after placement in structure, (2) to increase its resistance to decay, (3) to reduce its weight, (4) to improve its strength and mechanical properties, and (5) to prepare it for preservative treatments. Since the water will not run out of the wood, some means of evaporating it must be used. It must be understood, however, that it is impossible to eliminate all moisture from timber without scorching it; and, furthermore, that if the water were entirely removed the timber would reabsorb from the surrounding atmosphere and retain about 12 to 15 per cent of moisture.

According to H. D. Tiemann \* drying is much influenced by the following: (1) irregular shrinkage, (2) the different ways water is contained in the cells, (3) the manner in which it passes from center to surface, (4) the plasticity of the wood substance while in a hot and moist condition, and (5) the changes which are produced in the hygroscopic and chemical nature of the surface. The rate of drying is also affected by the ratio of the volume of the piece to its surface area. Logs with the bark left on dry very much more slowly and are more likely to decay than those without bark. End surfaces dry more rapidly than tangential or radial surfaces, but boards and long timbers lose most of their water through the sides. Within a given species, green timber of large moisture content dries in about the same length of time as that of lower moisture content. This is probably due to the fact that the sapwood, which contains most of the moisture, dries more rapidly than the heartwood. High temperatures soften the cell walls and promote the transfusion of moisture, thus favoring rapid drying.

Two methods of drying are practiced, air-drying or “seasoning” and kiln-drying. In air-drying the temperature and humidity conditions of the atmosphere exercise an important influence upon the rate of drying. Thus, owing to warmer atmosphere, ties and structural timbers cut

\* See Principles of Kiln Drying Lumber in *Lumber World Review*, Jan. 25, 1915.

in the summer dry at a much more rapid rate than those felled in the late autumn. In fact for many woods like oak and gum, timber felled in the summer often dries too fast and case hardens. For such timbers the cooler months with more humid atmosphere are a more favorable time for seasoning. A warm temperature and a free circulation of air are important requisites in air-drying. The latter consideration, however, is often overlooked. In experiments conducted by the Forest Product Laboratory\* it was demonstrated that ties piled, 7 by 2, 7 by 1 or 8 by 1, in single piles

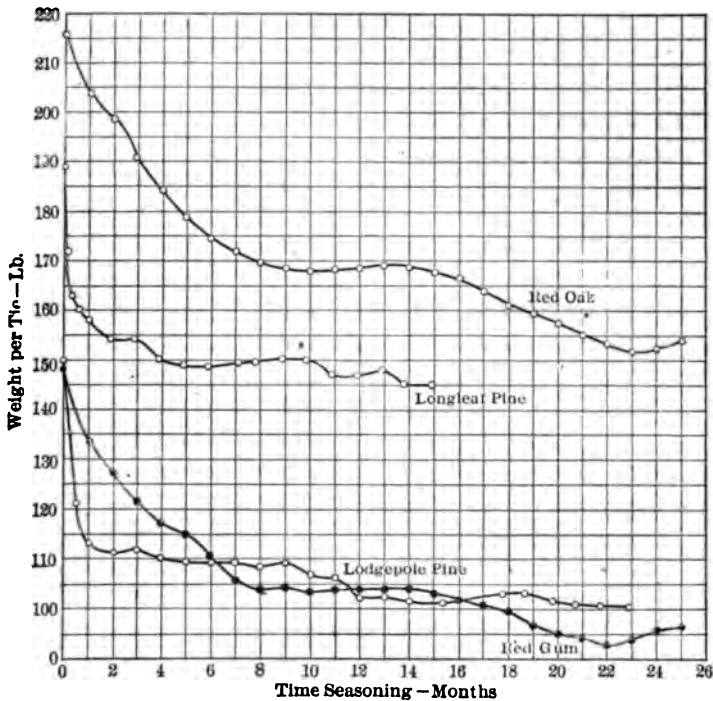


FIG. 6.—Losses in Weight of Ties with Long-continued Seasoning. (W. H. Kempfer in *Bull. No. 161, Am. Ry. Engr. Assoc.*, p. 215.)

dried more rapidly than those piled 9 by 9. Coniferous wood may be dried to constant weight in a shorter time and at a more rapid rate than many of the hardwoods. Fig. 6 shows drying charts for a number of tests on different kinds of ties. Large structural timbers of coniferous wood generally require seasoning for two summers, while smaller ones require only one summer in our northern latitudes. On account of the long time required for successful air-drying, a considerable proportion of certain woods like maple and gum rot before seasoning is completed.

\* *Bull. 118, Forest Service.*

There are a great many schemes in use for kiln-drying lumber, but in most of them either warm moist air or superheated steam is the drying medium. Kiln temperatures varying from room temperature to 180° F., and drying periods of a few days to several months are used. In some yards working on hardwoods, a short period of air-drying is followed by kiln-drying.

For most successful kiln-drying the timber should be brought to as high a temperature as it will stand without injury before drying is begun; otherwise the moisture in the hot outer fibers of the wood will tend to flow toward the cooler interior. The proper condition may be obtained by circulating air with a high humidity until the wood is thoroughly heated, and then gradually diminishing the humidity to bring about drying. Tiemann\* states that oak, western larch and cypress require a high humidity (80-90 per cent), at the start and should be held at 50 per cent until toward the end of the run. Most of the conifers may, however, be run at lower humidities. With some green timbers initial temperatures of only 120° F. or even less must be used, while others can be started at temperatures above 212° F. Besides control over humidity and temperature, it is essential that the uniformity of both be secured by ample circulation.

Air-drying is less expensive in operation but ruins more lumber than kiln-drying. Proper air-drying is preferable, nevertheless, to badly managed kiln-drying. It is used very extensively in drying ties and the larger sizes of structural timbers. With kiln-drying there is a smaller loss in timber, usually less than 10 per cent even in timbers like gum. Also, with kiln-drying, the wood is more thoroughly and evenly dried, thus reducing the hygroscopicity of the wood. It is claimed, furthermore, that sap stains may be prevented and the gums and resins fixed by correct kiln-drying.

The figures in the following table † furnish an approximation of the amount of water lost in drying green timber:

POUNDS OF WATER LOST IN DRYING 100 POUNDS OF GREEN WOOD  
IN THE KILN

Common Names of Species.	Sapwood or Outer Part.	Heartwood or Interior.
(1) Pines, cedars, spruces, and firs.....	45-65	16-25
(2) Cypress, extremely variable.....	50-65	18-60
(3) Poplar, cottonwood, basswood.....	60-65	40-60
(4) Oak, beech, ash, elm, maple, birch, hickory, chestnut, walnut, and sycamore.....	40-50	30-40

The lighter kinds have the most water in the sapwood; thus sycamore has more than hickory.

\* See previous citation.

† Roth in *Bulletin* No. 10, U. S. Dept. of Agric.

**161. Shrinkage and Its Effects.**—For purposes of illustration consider a very small, thin transverse section of green wood, Fig. 7A. If this section is very slowly and uniformly dried no change will be noted in the disc until the water in the pores is evaporated. Then the cell-walls will gradually become thinner and the sides of the disc *ab*, *bd*, etc., will shorten. No contraction in length of the disc is, however, observable. Furthermore, since the thickness of the end walls of the cells or fibers is very small compared to their length, it is apparent that longitudinal shrinkage of a thicker disc composing several fiber lengths will be negligible. If we repeat the experiment with a disc like that of Fig. 7B, we

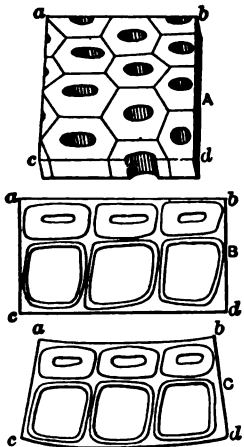


FIG. 7.—Warping of Wood.

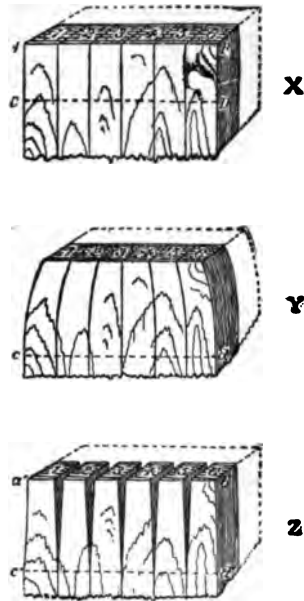


FIG. 8.—Formation of Checks.

will observe that the side *ab* shortens more than *ed* and that the surfaces *ab* and *cd* are curved, Fig. 7C. In other words, thick-walled cells shrink more than those having thin walls. We shall, therefore, find that a curved disc of wood one annual ring in width will straighten in drying owing to the fact that the thick-walled cells of summer wood shrink more than the thinner walled cells of spring wood. This inequality in shrinkage between the various cells produces stresses of a serious nature during the drying of timber.

Again, if a stream of warm air is directed against the side *cd* of the moistened disc, Fig. 7A, it will be noted that it shortens much more rapidly than *ab*, owing to the more rapid evaporation of moisture. When all portions of the disc are equally dry, *ab* and *cd* are again of equal length.



Thus a partially dried board exposed to the sun's rays becomes concave on the upper side, but may be straightened by turning the board over and allowing the moisture in the convex surface to be evaporated.

Since water is evaporated more rapidly from the ends of the wood elements than from the sides, a piece of wood like that in Fig. 8X will shrink more laterally at *ab* than at *cd*. This action produces bending in the piece as shown in Fig. 8Y. If the rapidity of drying is sufficiently great, the resulting pull across the grain of the wood will exceed its tensile strength and checking ensues, Fig. 8Z. After the piece has completely dried

many of these checks close, although the weakening effect still remains. Rapid drying of the outside logs and timbers often causes similar cracks to appear on the longitudinal surfaces. Not all of these radial cracks close when seasoning is complete; some gradually open and remain a permanent source of weakness for a reason which we shall now explain.

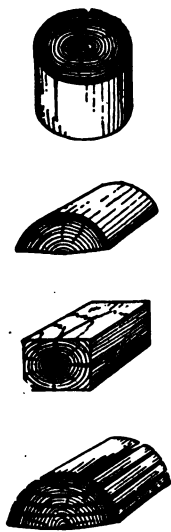


FIG. 9.—Effects of Shrinkage.

Looking at Fig. 14, we note that the cells in medullary rays are elongated in a direction perpendicular to the longitudinal elements. Since cells in the rays obey laws of shrinkage similar to those governing the behavior of the longitudinal elements, it is at once apparent that the rays will shrink in the longitudinal and tangential direction but not radially, whereas the longitudinal elements will shrink radially and tangentially but not longitudinally. Consequently, the lateral shrinkage of the ray in the longitudinal direction will be hindered by the adjoining longitudinal elements and the length of the ray will be shortened by the radial shrinkage of the longitudinal elements. The mutually perpendicular

tensile and compressive forces thus produced are often sufficient to break the bond between the ray and the adjacent longitudinal fibers. Once the bond is broken, further circumferential shrinkage operates to widen rather than to close the breach.

In woods like oak, where the number of pith rays is very large, it is probable that the slight longitudinal shrinkage is due principally to lateral shrinkage of the rays. Also, the resistance offered by the rays to oppose radial shrinkage of the wood and the great shrinkage of the summer wood are the reasons why the tangential shrinkage of wood is always more than the radial shrinkage. The difference in shrinkage in these two directions is the cause of much difficulty in drying. Besides producing the checks in logs and sawn timbers, this difference between the tangential and radial shrinkage also causes the flat surfaces of a sawn log to become convex, as shown in Fig. 9.

When hardwoods, like oak, are quickly dried the water is evaporated more rapidly from the outside than it can be brought to the surface. As a result the outer portion of the piece checks; or, if it has sufficient plasticity under the influence of heat and moisture, the surface may take a set and harden just as wood bent in steam retains its shape after drying. If drying continues, the interior of the stick shrinks and the circumferential tension in the outer shell is relieved. Further shrinkage of the piece,

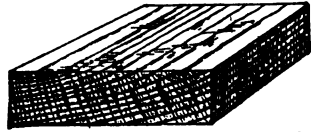


FIG. 10.—“Honeycombed” Board. The checks or cracks form along the pith-rays.

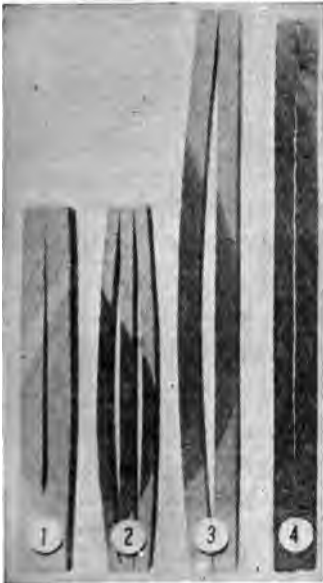


FIG. 11.—Illustrating the Elimination of Case-hardening in Kiln Dry Red Gum by Steaming at the End of the Drying Period. No. 1 sawn after no final steaming; Nos. 2 and 3 after eighteen minutes final steaming; No. 4 after thirty-six minutes final steaming. (J. E. Imrie, before Gum Lumber Mfrs. Assoc., Jan. 16, 1916.)

as a whole, is diminished by the rigidity of the outer shell. This brings about a gradual reversal of stress in the shell and causes radial tension in the interior. If the radial tension becomes excessive, rupture will occur, as shown in Fig. 10. This phenomenon is called *case-hardening* or *honey-combing*. It may exist in lumber without the cracks being noticed, but is revealed immediately upon sawing. A simple test for case-hardening is that used by Tiemann (see Fig. 11).

Rapid drying of green cedar and redwood at high temperatures often produces a collapse of the cell walls. This is brought about by the radial tensile stresses produced on the cell walls by the withdrawal of free water. Such defects lower the strength of the timber; they may be avoided by using lower initial temperatures in drying.

Dried pieces of wood greedily absorb water with an increase in volume until the cell walls are saturated, the *fiber-saturation* point is reached;\* subsequent filling of the lumen in the cells is accompanied by no further swelling, Fig. 12.

The following average values for per cent moisture at fiber-saturation point

\* Tiemann reported in *Proc. Soc. Am. For.*, Vol. 8, p. 313, that blue gum is an exception to the above rule, since it “begins to shrink immediately from green condition, even at 70 to 90 per cent moisture.”

were obtained by Tiemann (*Cir. No. 108*, Forest Service). Each result represents 40 or more tests per species of wood. In some cases individual values varied as much as 10 per cent from the averages.

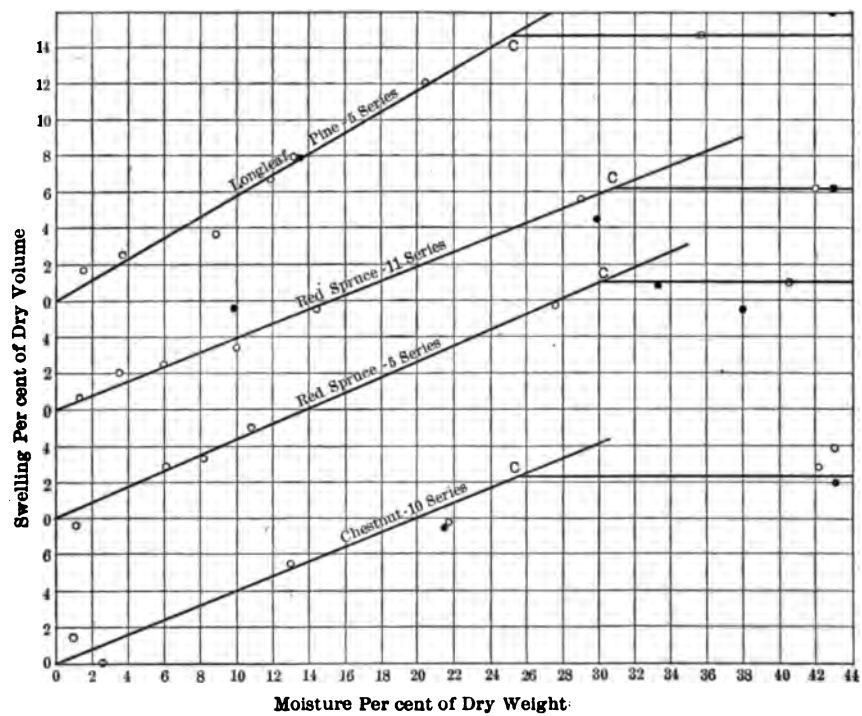


FIG. 12.—Relation between Swelling and Moisture. Each point is the average of from five to eleven specimens. Black dots indicate specimens that were kiln-dried and then allowed to reabsorb moisture. The fiber-saturation point is at C. (*Cir. 108* of Forest Service.)

Species.	Per Cent.	Species.	Per Cent.
Longleaf pine . . . . .	25	Red Spruce . . . . .	31
Loblolly pine . . . . .	24	Chestnut . . . . .	25
Norway pine, heartwood . . . . .	30	Tamarack . . . . .	30
Norway pine, sapwood . . . . .	28	Red gum . . . . .	25
Douglas fir . . . . .	23	White ash . . . . .	20

Repeated wetting and drying weakens timber, causes expansion and contraction, and, in addition, produces conditions which promote decay. Timber must, therefore, be protected from moisture, if constancy in volume and its life are to be conserved. The swelling of wooden pipes and tanks after water is admitted often produces large stresses in both bands

and timber. The amount of swelling and the stresses caused by it should be given careful consideration in design.

**162. Amount of Shrinkage.**—As a rule, if the density of the sapwood and heartwood are the same, the former will shrink more than the latter; but heavy heartwood shrinks more than light sapwood. Coniferous woods like pine, spruce, cedar, cypress and redwood shrink uniformly and do not check much in drying. Oak, beech, chestnut, elm, hickory, gum, and other hardwoods shrink considerably and check more or less depending on the care exercised in drying.

In general, the radial shrinkage of wood is about 60 per cent of the tangential, and the longitudinal shrinkage is negligible. Therefore, the volumetric shrinkage is practically 1.6 times the tangential shrinkage. Elaborate experiments by the Forest Products Laboratory show that the shrinkage in either direction varies as the first power of the density and that this relation holds for all species. The following coefficients worked out by Mr. J. A. Newlin give the per cent shrinkage when multiplied by the specific gravity: 26.5 for volume, 9.5 for radial, and 16.5 for tangential shrinkage.

Approximate values of the tangential shrinkage of air-dried material are given below, more accurate results appear in Table 1, Ch. VI.

APPROXIMATE SHRINKAGE OF A BOARD, OR SET OF BOARDS, 100  
INCHES WIDE, DRYING IN THE OPEN AIR

Common Names of Species.	Lateral Shrinkage Inches.
(1) All light conifers (soft pine, spruce, cedar, cypress) . . . . .	3
(2) Heavy conifers (hard pine, tamarack, yew), honey-locust, box-elder, wood of old oaks . . . . .	4
(3) Ash, elm, walnut, poplar, maple, beech, sycamore, cherry, black locust . .	5
(4) Basswood, birch, chestnut, horse chestnut, blue beech, young locust . . .	6
(5) Hickory, young oak, especially red oak . . . . .	Up to 10

PRINCIPAL NATIVE WOODS \*

**163. The sources, characteristics, and uses** of the more important woods will now be briefly described. In the list which follows, an attempt has been made to arrange the various classes of wood in order of economic importance. Within a given class, however, there may be individual species which are of less value than species in a class farther down in the list. For example: Sugar pine, lodgepole pine, and tupelo gum are less valuable than hickory; but the pines and gums as groups are more valu-

\* Compiled largely from Hough's *Handbook of Trees*, Snow's *Principal Species of Wood*, and *Bulletin No. 232* of the U. S. Dept. of Agric.

able than hickory. The mechanical properties of many of these woods can be found in Ch. VI.

**164. Southern Yellow Pine** is the term applied to the species of yellow pine which are found in Southern States from Virginia to Texas; most of our supply now comes from Louisiana, Mississippi, Texas and North Carolina. Included in this group are longleaf, shortleaf, loblolly, Cuban and pond pine. Difficulty experienced in separating these species has brought about the use of this inclusive term and caused the adoption of grading rules for quality classification. Longleaf and Cuban pine are, in general, heavy, hard, tough, stiff and very strong woods. They are highly durable in dry localities and fairly so in contact with the ground. The other members of the southern yellow pine group are more variable. Most specimens of the latter are inferior in structural properties to longleaf, but they generally possess good strength and durability when not in contact with the ground. The southern yellow pines are the most important source of dimension timber for all heavy construction. They also provide much lumber for joists, posts, piling, and building construction. When treated with preservatives the harder woods of this group make good ties and paving blocks.

**165. White Pine** is still found to a limited extent in the States north of the Ohio River and east of the Dakotas, most abundantly in Minnesota, Wisconsin and Maine. A somewhat inferior grade of white pine is obtained along the Rocky Mountains. It is a soft, uniform white wood which shrinks very little in seasoning, works easily, nails without splitting, and takes paint well. It is not very strong but quite durable. For window sash, interior trim and pattern making, the demand for this wood is very great.

**166. Norway Pine** is now found principally in the States bordering on the Great Lakes. It is a light wood of fair hardness and strength, but is not durable in contact with the ground. Some dimension timber, masts, spars, piling and interior trim are made of it. It is often sold for white pine.

**167. Western Yellow Pine** grows on the eastern slopes of the Rockies and in country westward to the coast. It is lighter, softer, weaker and less durable than longleaf pine, but heavier and stronger than white pine. It is considerably used for dimension timbers, ties and mine timbers; although unless treated, it is better fitted for trim and pattern making.

**168. Sugar Pine**, found in California and Oregon, is a light soft wood resembling white pine; it has similar uses.

**169. Lodgepole Pine**, a timber found from the Rockies to the Pacific Coast, is a light, brittle, straight-grained wood of low strength. It is also difficult to season. This pine is used principally for poles, posts and ties.

**170. White Oak** of commerce includes true white, post, burr, overcup, swamp white, cow and chestnut oaks. Oak of this class may be found in the States east of Colorado, but the principal supply comes from the Virginias, Tennessee, Arkansas, Kentucky, Ohio and Missouri. These oaks are all hard, heavy, strong, tough, dense woods which are durable in contact with the soil. They shrink considerably and are likely to check in seasoning. The wood is capable of receiving a high polish. White oak is much used for furniture, cross-ties, agricultural implements, fence posts, wagon-stock, cooperage and baskets. These oaks are the most valuable of the hardwoods.

**171. Red Oak** of commerce includes red, pin, Spanish and black oaks. The sources of supply are the same as for white oak. The wood of the red oaks, though hard and strong, is more porous, somewhat lighter and weaker, and far less durable in contact with the soil than white oak. It is used chiefly for interior finish and furniture. It is easily impregnated with preservative and when so treated makes excellent cross-ties.

**172. Live Oak** is found along the coast of the southern Atlantic and Gulf States in California and Oregon. The wood is very heavy, hard, tough, strong, durable and difficult to work. It is used for implements, wagons and in ship building.

**173. Douglas Fir** is grown along the Pacific Coast, the most valuable forests being in Oregon, Washington and British Columbia. The wood is strong, brittle and fairly durable. It is the best structural timber of the northwest. This clear, straight-grained wood is widely used for building construction, dimension timber, ties, piles, boats, paving blocks, tanks, conduits and furniture.

**174. Hemlock** is found in the Great Lakes States, in southeastern Canada and from Maine to Georgia along the Appalachian range. A Western hemlock grows on the Pacific Coast from Northern California to Alaska. The wood is light, soft and brittle. The Western variety is moderately strong and fairly durable, but Eastern hemlock is weak and not durable in contact with the ground. Hemlock holds nails well and is much used in house framing. Western hemlock is considerably used for dimension timber and cross-ties.

**175. Spruce.**—Red spruce is found principally in New York, New England and West Virginia; white spruce in nearly all parts of Central Canada and in the Great Lakes States; Sitka spruce in Washington, Oregon and Idaho. The woods of these species are light and soft. They have low strength and fair durability. Spruce is used chiefly for paper pulp, railway ties, resonance wood, piles, aeroplanes and lumber.

**176. Cypress** grows along the eastern and Gulf coasts from Maryland to Texas and along the Mississippi Valley as far north as Illinois. Louisiana, Florida and Georgia are the chief producers. It is a light,

soft wood of medium strength which is rather difficult to season but very durable. Cypress is used for siding, shingles, sash, doors, tanks, silos and railway ties.

**177. Hard (Sugar) Maple** grows in all of the States east of Colorado but most abundantly in the Great Lakes region. The wood is heavy, tough, hard and strong, but not durable. The grain is often curly or has "bird's eyes." It is used for interior finish, flooring, furniture, ship and car construction. When treated this wood may be used for cross-ties.

**178. Soft (Red) Maple** is found in the region with hard maple. The wood is heavy, hard and strong, but inferior to hard maple. It is fairly easy to work and is used for furniture, cabinet making, turnery and gun stocks.

**179. Chestnut** grows on both slopes of the Appalachian Range, but is produced principally in the Virginias, Pennsylvania, Tennessee, and Connecticut. It is a light, soft, weak and brittle wood which is very durable in contact with the ground. The wood shrinks considerably and checks in seasoning, but works easily. Chestnut is much used for fence posts, poles and cross-ties; also for exposed constructions, furniture, and cooperage.

**180. Red (Sweet) Gum** grows in the same regions as cypress and is supplied most abundantly from Arkansas and Mississippi. The wood is not durable in the ground, soft, rather brittle and of moderate weight and strength. It is easily worked, but warps and twists in seasoning. When highly polished it makes attractive furniture and interior trim. Other uses are for flooring, slack cooperage, turnery and wagon stock.

**181. Tupelo (Sour) Gum** is found with red gum, but is most abundant in the Gulf States. It has about the same weight and strength as red gum but is tougher. This wood also is difficult to season and finds more or less use in the manufacture of boxes, furniture, wagon boxes, flooring and finishing.

**182. Hickory** is fast disappearing in this country; the present supply is obtained from Arkansas, Tennessee, and the Ohio basin. The more abundant varieties of hickory furnish very heavy, hard wood which is stronger and tougher than other native woods. Hickory checks and shrinks largely in seasoning and is difficult to work. It is subject to insect attack and not durable. The chief uses are for wagon-stock, agricultural implements, axe-handles, hoops and baskets.

**183. Yellow Poplar (Whitewood)** is found in nearly all States east of the Mississippi River and south of the Great Lakes; it is gotten from the Virginias, Tennessee and Kentucky principally. The wood is light, soft, brittle, weak and easy to work. It shrinks considerably, but holds nails well and is fairly durable. Whitewood is a very valuable wood for

interior finish, furniture, shelving, drawers, wagon-bodies and boxes; it is also used for siding and paneling.

**184. Basswood** is scattered over the eastern half of the United States with the exception of the Southern Atlantic and Gulf Coasts; Wisconsin, Michigan, West Virginia and New York lead in production. The wood is soft, light, weak, brittle and not durable. It shrinks considerably, but is very uniform and works easily. Although slightly inferior to whitewood, it is used for similar purposes.

**185. Redwood** grows abundantly along the coast of California. The wood is light, soft, straight-grained and very durable. In the West it is used for all kinds of lumber; ties, shingles, poles, paving blocks, tanks and conduits.

**186. Yellow and Sweet Birch** are found in the region east of the Mississippi River and north of the Gulf States, also in Southeastern Canada; Wisconsin, Michigan and Maine lead in production. The wood is heavy, hard, stiff, strong, and tough; but it is not durable when exposed. In view of its hardness, it works easily and takes a high polish. Birch is much used for interior finish, furniture, turnery and carving.

**187. Larch or Tamarack.**—The eastern variety of this wood, generally called tamarack, is abundant in the Great Lakes region, New England, northern and eastern Canada. Western larch is found principally along the Columbia River Valley. The Western variety is of medium weight, rather tough, hard and durable, but somewhat less strong than Douglas fir or Western hemlock. It is used for lumber, lath, cross-ties, poles and slack cooperage. Tamarack is slightly less heavy and strong than Western larch. Its uses are similar.

**188. Ash.**—Varieties of ash are found in nearly all States east of the Rockies. Black ash is confined to the Northern States of this region, but the white and green species are widely found. The wood of the white and green ashes is heavy, hard, strong and fairly tough. It is straight grained, shrinks little in seasoning and can be polished. It is used for finishing lumber, in wagon construction, farm implements, furniture and cabinet work. Black ash makes a lighter, inferior wood to that of the white or green ashes. It is used as a substitute for them and in basket making.

**189. Red and White Cedar.**—White cedar is found along the eastern coast and around the Great Lakes; red cedar grows in the region east of Colorado and north of Florida. Western red cedar is grown largely in Washington, Oregon, Idaho and Montana. Cedar wood is very light, soft, weak and brittle. Its low shrinkage and great durability, when exposed, make cedar valuable for shingles, siding, posts, poles and ties. Red cedar is much used for moth-proof chests.



**190. Beech** grows in the region east of the Mississippi and in southeastern Canada. The wood is heavy, hard, strong, tough, but not durable in contact with the soil. It shrinks and checks considerably in seasoning. It is used for furniture, plane-stocks, handles and shoe-lasts.

**191. Elm.**—White and slippery elms grow in the States east of Colorado; rock elm is found largely in Michigan and the States bordering on the Ohio River. The wood of the slippery and rock elms is heavy, hard, strong, tough, durable and difficult to split. The wood of white elm is somewhat inferior to that of the rock and slippery elms. Elm wood is used for agricultural implements, wheel-stock, boats, furniture, cross-ties, posts, and poles.

**192. Cottonwood** is found scattered over the region east of the Rockies, excepting in Maine; abundantly in Arkansas, Mississippi, Louisiana, and Tennessee. It is a soft, weak wood similar to whitewood but inferior in quality. It is considerably used for slack cooperage, fencing and paper pulp.

**193. Black Walnut** is found over the eastern half of the United States with the exception of Southern Atlantic and Gulf Coasts; Indiana, Ohio and Missouri are principal producers. Some varieties are obtainable from New Mexico, Arizona and California. The wood is heavy, hard, strong and easily worked. It has a dark chocolate color and is susceptible of a high polish. Owing to scarcity, its usage is confined largely to making of cabinets, furniture and gun-stocks.

**194. Sycamore** is most abundantly grown in the Ohio and Mississippi basin, although common in most States east of Colorado. The wood is of medium weight, hardness and strength. It is rather brittle, difficult to work, and liable to check and warp in seasoning. Sycamore makes a pleasing appearance when quarter-sawn. It is used for interior trim, cabinet making, tobacco boxes, and cooperage.

**195. Eucalyptus** is a rapidly growing Australian tree of which a large number of varieties have been transplanted in California. Blue gum (*Eucalyptus Globulus*) is the most important of these. This durable wood is very heavy, hard, tough, and strong, comparing favorably in these respects with hickory. It is, however, extremely difficult to season, since it checks and warps very badly. From results obtained in Australia, it is predicted that the American blue gum will furnish a satisfactory wood for cross-ties, fence-posts, poles, pilos, paving blocks and wagon-stock.

**196. Catalpa** is grown in Mississippi, Alabama, Georgia, and Florida; and hardy catalpa in Missouri, Illinois, Indiana, Kentucky and Tennessee. The wood of the two species is similar, being light, soft, and weak. It is very durable in contact with the ground and makes excellent fence posts and poles. If well protected with tie-plates it also serves for cross-ties.

very rapid growth which is characteristic of the tree has led to the ting of it for such purposes.

#### THE IDENTIFICATION OF WOODS \*

**The Microscopic Structure of Wood.**—Thus far we have considered the structure of wood which is readily discerned by the naked eye. With a microscope it has been ascertained that a few definite types of cells and fibers form the structures of all woods. Inasmuch as the recognition of these types of cells, as well as their arrangement and condition is an important aid not only in identifying species, but also in accounting for mechanical properties, a brief account of them is made. There are four main types of these microscopic elements which, with numerous transitional forms, make up the structure of timber. These are (1) *tracheids*, (2) *parenchyma*, (3) *vessels*, (4) *wood fibers*. The two first-mentioned constitute the wood of the conifers, but all four types are found in varying amounts in broad-leaved trees.

*Tracheids* are slim polygonal cells provided with tapering ends. They are small and of little importance in the hardwoods; but in the conifers where they form the main constituents of the wood they range from 0.05 to 0.35 in. in length with a diameter of one-fiftieth to one one-hundredth of the length. The side walls of the tracheids are perforated with *bordered pits* which are funnel-like depressions most thickly found near the ends of the cell (see Fig. 13). Through the thin walls at the bottoms of these pits the sap flows from one cell to another. In Douglas fir, spiral ridges are found on the inside of the tracheids, while in long leaf and Norway pine the ray tracheids have irregular dentations on the inner surfaces.

*Parenchyma* are subordinate elements, which, like the tracheids, may be arranged end to end in a vertical line, thus forming the wood parenchyma fibers; or grouped in bundles with their long axes extending radially, they compose the entire pith rays of the hardwoods and the main part of the rays in softwoods. Sometimes, as in white oak, chestnut and hickory, the wood parenchyma fibers are arranged parallel to the vessels in the rings and appear as fine concentric lines on the cross-section. In some woods they form the boundaries of the rings; in some they are scattered through the wood irregularly; in others they are arranged in radial planes, and in still others they surround the larger vessels. Parenchyma are minute, thin-walled elements tapering at the ends and subdivided by transverse walls into short, prismatic cells. The side-walls of parenchyma are dotted with minute cylindrical depressions called *simple pits* as in Fig. 13F.

\* For a more complete discussion of the microscopic structure and the identification of woods, the reader is referred to Record's *Economic Woods* and Hough's *American Woods*.

By pressure of large bordering vessels the parenchyma running vertically are sometimes flattened into the conjugate form shown in Fig. 13H. In oak, hickory and walnut the individual cells of the vertical parenchyma are often separated by crystals of calcium oxalate, see Fig. 13G.

*Vessels* are small pipe-like elements of indefinite length, the walls of which are covered with bordered pits (Fig. 13K). The diameter of vessels is quite variable, sometimes reaching 0.02 in oak, but more often it is less than 0.01 in. While growing, constrictions are produced in the side-

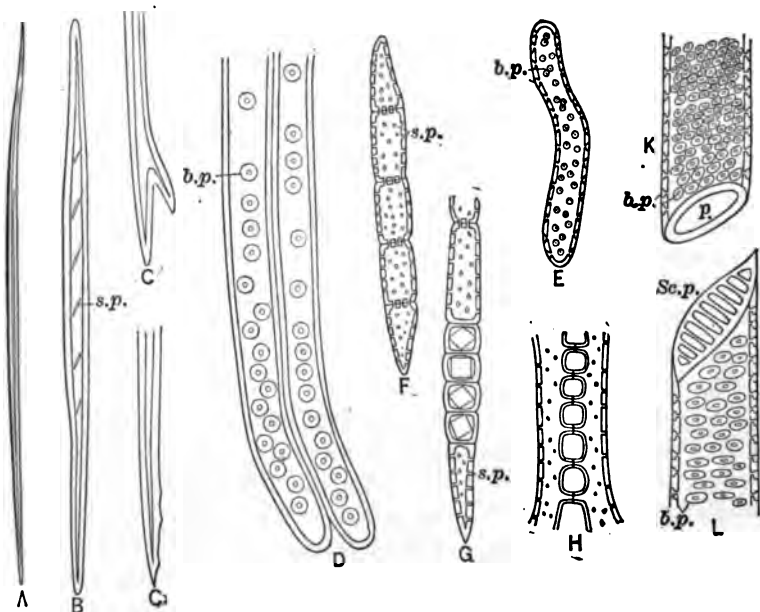


FIG. 13.—Types of Wood Cells.

A, wood fiber with narrow lumen; B, wood fiber with wider lumen and simple pits (s. p.); C, wood fiber with saw-toothed end; C', wood fiber with forked end; D, tracheids with bordered pits (b. p.) from pine; E, tracheid from oak; F, wood parenchyma fiber with individual cells and simple pits (s. p.); G, wood parenchyma with crystals of calcium oxalate from walnut; H, conjugate parenchyma cells; K, part of segment of a vessel with simple perforation (p.); L, part of segment of a vessel with scalariform perforation (Sc. p.). All much enlarged. (After Record.)

walls, thus indicating the segments of growth. These segments may fit together (1) in a perfect transverse plane, (2) in an oblique plane, or (3) as in oak and gum, the faces of the segments may be oblique and have blind ends extending beyond the main line of constriction. In type (1) the opening from one segment to another is round, but in (2) and (3) the perforations may be scalariform as in Fig. 13L. Vessels in the sapwood serve as vertical water supply lines for the growing portion of the tree, but in the heartwood they are frequently clogged with sac-like protrusions from adjacent parenchyma cells. These protrusions are called *tyloses*.

**Wood fibers** are thin elongated cells tapering to a point at either end. They have thick walls which are ordinarily indented by inclined slit-like simple pits, see Fig. 13B. In mahogany the fibers are divided by cross partitions (septate fibers); in other woods the fibers are often forked at the ends (Fig. 13C'), a condition which decreases the cleavability of the wood. Wood fibers vary from a fiftieth to a tenth of an inch in length. They are found most commonly in the central portion of the annual rings of the hardwoods and are an important source of strength, toughness and hardness.

As mentioned before these various types of fibers grade into each other by transitional stages. Thus the wood fibers exhibit forms approaching the tracheids in some woods and approximating wood parenchyma in others, and the tracheids sometimes grade into the vessels.

**198. The Structure of Coniferous Woods.**—In Fig. 14 are shown (1) a sector of spruce in natural size and (2) a part of one ring from the same magnified 100 times but oriented to correspond with piece (1). Looking at the upper face of piece (2) we are at once impressed by the regular arrangement of the tracheids in radial rows. From left to right they become flattened radially showing the increase in density in passing from the spring wood to the summer wood. On the lower front portion of this piece we notice also the bordered pits, of which enlarged types are shown at *a*, *b* and *c*.

The dark lines in piece (1) represent the medullary rays of which five are exposed on the right face of piece (2). One of these rays is seen in section on the front face of piece (2). It will be noted that the cells in the rays are elongated radially and that each ray is one cell wide (uniseriate) and several cells deep. In this wood, the top and bottom cells of each ray are tracheids (*n*), but the intermediate cells are parenchyma (*m*).

The above example is typical of the regularity of arrangement of the cell structure in coniferous wood. In pine, spruce, Douglas fir, and tam-

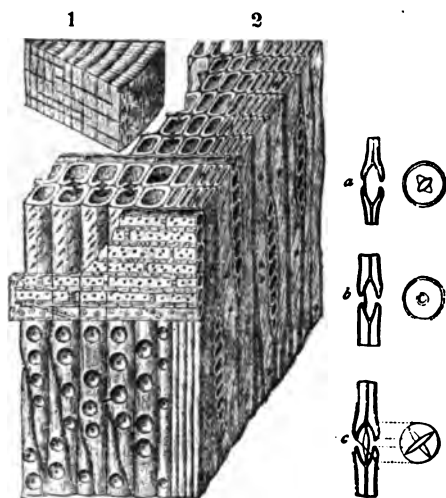


FIG. 14.—Wood of Spruce. 1, natural size; 2, small part of one ring magnified 100 times. The vertical tubes are wood-fibers, in this case all "tracheids." *m*, medullary or pith ray; *n*, transverse tracheids or pith-ray; *a*, *b*, and *c*, bordered pits of the tracheids, more enlarged.

arack vertical and radial resin ducts are found, which are interconnected here and there to permit the passage of resin. These ducts are often large enough to be distinguished by the naked eye. They are long canals bounded by groups of thin-walled cells, termed epithelial cells. Fig. 15 shows a cross-section of shortleaf pine which passes through a vertical resin duct.

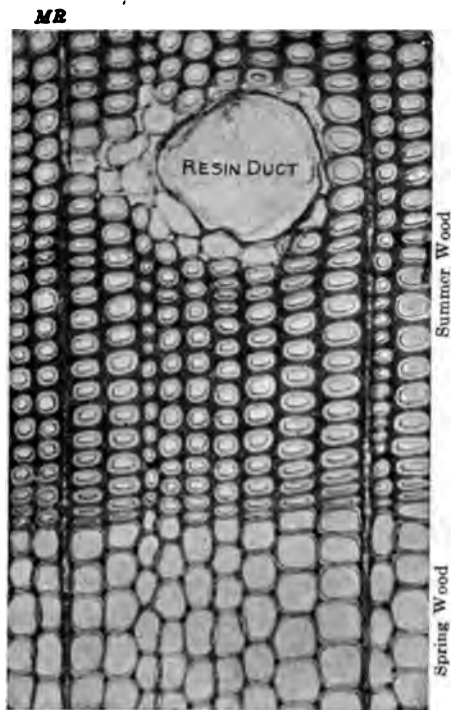


FIG. 15.—Cross-section of Shortleaf Pine, Showing Resin Duct Surrounded by Epithelial Cells. (*Bul.* 101, U. S. Dept. Agric., Pl. 1. Magnification = 125 diam.)

Radial resin ducts are commonly inclosed in multi-seriate rays. Such ducts may often be formed by injury; chipping the outer sapwood of longleaf pine opens the resin ducts and affords a method of securing turpentine and allied products.

Tyloses are sometimes found in tracheids adjoining parenchymatous cells, but in conifers they are more often noticed in the resin ducts. Norway pine, Western pine, white pine, and sugar pine, are the main coniferous woods which have abundant tyloses (see Art. 208).

**199. The Structure of Wood from Broad-leaved Trees.**—As we have previously mentioned, the arrangement of the cell elements in the wood of the broad-leaved trees is far more varied and complex than in the conifers. We shall consider two examples, red oak to illustrate ring-porous woods and sugar maple to illustrate diffuse-porous woods.

The magnified cross-sectional view of red oak in Fig. 16 shows very clearly the irregular grouping of the large vessels in the spring wood, with a more or less gradual transition to smaller ones scattered here and there through the summer wood. In good oak these vessels occupy less than 10 per cent of the volume of the wood, but in poorer varieties they may amount to 25 per cent. In the middle of the annual ring the dark, solid looking patches represent the cut ends of the wood fibers. The thicker the walls of these fibers and the greater the proportion of them in the wood, the stronger and tougher will it be. In good timber the fibers constitute one-half of the volume. Two medullary rays are also exposed

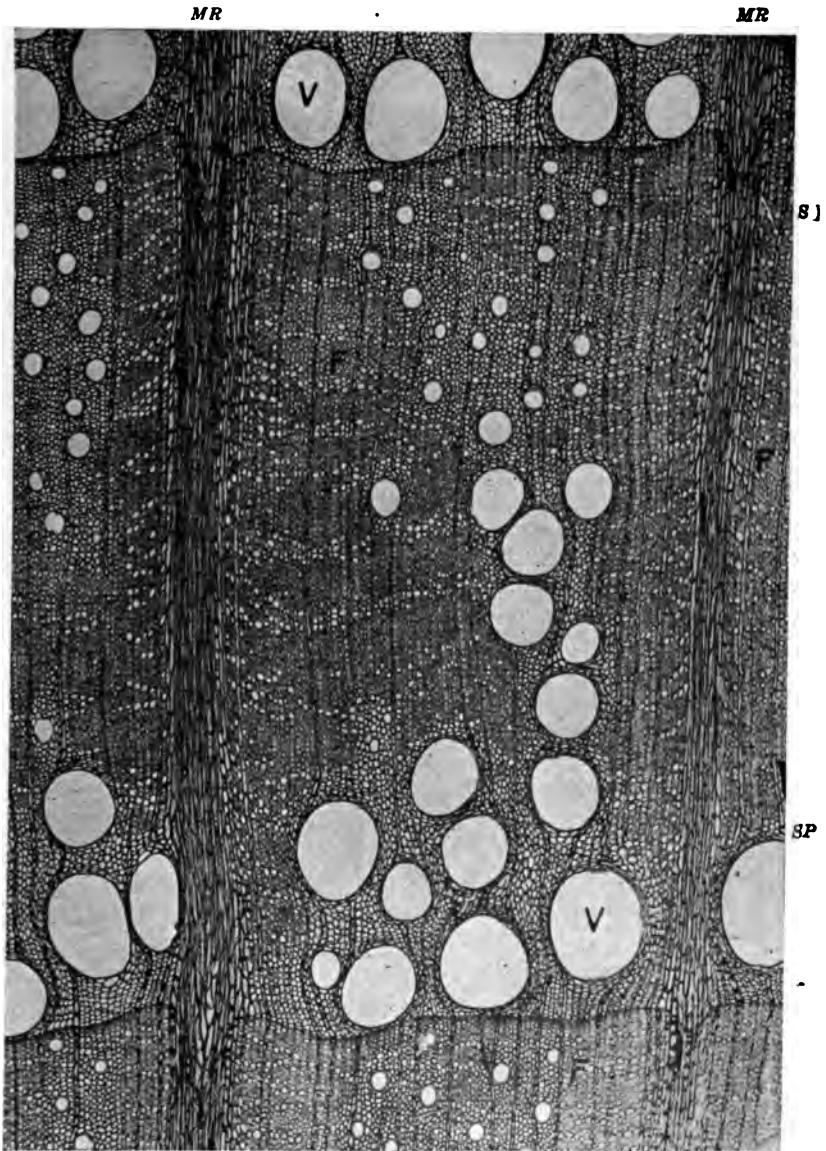


FIG. 16.—Photomicrograph of Cross-section of Red Oak, a Ring-porous Wood.

*SP*, spring wood (note open pores); *S*, summer wood; *F*, fibers; *V*, vessels or pores; *MR*, medullary ray. (Prepared by the Forest Products Laboratory Forest Service, U. S. Dept. of Agric. Magnified 40 diameters.)

in this view. The width of the band of cells composing these rays is a decided contrast to the uniseriate rays of the conifers. In the oaks, rays are often a hundred cells in width and an inch or more in height. It will be observed, however, that they always taper in width, at the top and bottom, to a single cell. Fig. 17 brings out the comparative size of the medullary rays and the ring-thickness in oak. All of the cells in the rays of the dicotyledons are parenchyma. Some of the individual parenchyma can be distinguished in Fig. 16. Beside the medullary rays of large size other uniseriate pith rays of parenchymatous cells may also be seen in Fig.

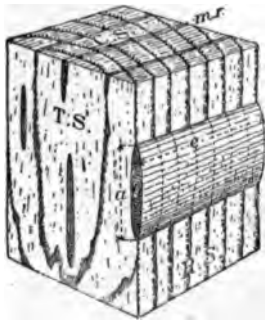


FIG. 17.—Block of Oak. *CS*, cross-section; *RS*, radial section; *TS*, tangential section; *mr*, medullary or pith ray; *a*, height, *b*, width, and *e*, length of a pith-ray.

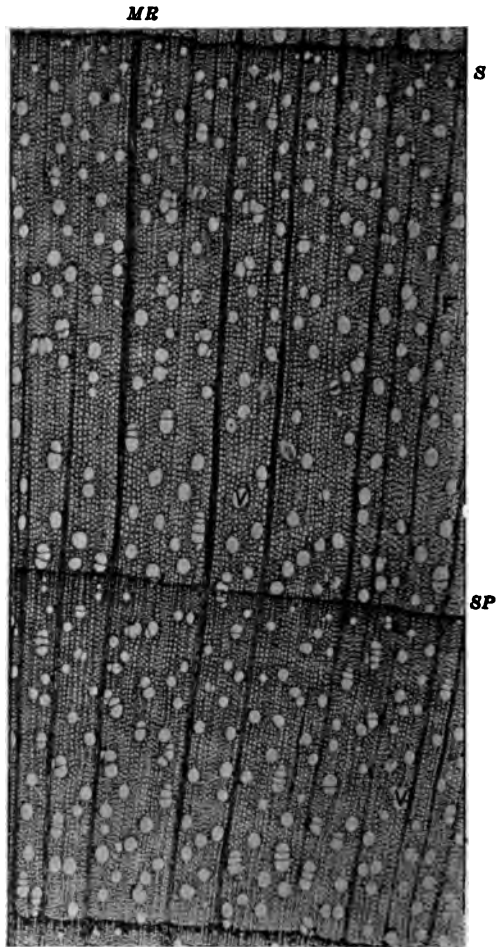


FIG. 18.—Cross-section of Hard Maple.

(Note tendency of pores, *V*, to form radial groups. Spring wood, *SP*, is much like the summer wood, *S*. Narrow medullary ray, *MR*. Fibers, *F*. Magnification = 34 diameters. (Photo prepared by Forest Products Laboratory, Forest Service, U. S. Dept. Agric.)

16. The total proportion of rays in good white oak generally lies between 15 and 25 per cent.

The arrangement of pores in a ring-porous wood like the sugar maple is more uniform, the diameters are smaller, and the variation in size is less

than in the oak (Fig. 18). Scarcely any difference is to be noted in the size of the vessels in the spring wood and summer wood, but there is a tendency toward radial grouping of two to four cells which is plainly marked. The medullary rays are much narrower than in the oak but broader than in the conifers. When the wood is quarter-sawed these rays produce a silvery appearance. There are, however, numerous intermediate pith rays of one-cell width in evidence. A further distinguishing feature of this wood is the distinct markings which limit the growth rings.

Tyloses are abundantly found in the vessels of the following hardwoods: in white, Garry, over-cup, bur, swamp, cow, valley and post oaks, in most hickories, in chestnut, black locust, and osage orange. Tyloses apparently increase the resistance of the wood to the decay and also decreases the penetrance to preservatives.\*

**200. The Use of a Key in Distinguishing Woods.**†—Nobody need expect to be able to use successfully any key for the distinction of woods or of any other class of natural objects without some practice. This is especially true with regard to woods, which are apt to vary much, and when the key is based on such meager general data as the present. The best course to adopt is to supply one's self with a small sample collection of woods accurately named.‡ Small, polished tablets are of little use for this purpose. The pieces should be large enough, if possible, to include pith and bark, and of sufficient width to permit ready inspection of the cross-section. By examining these with the aid of the key, beginning with the better-known woods, one will soon learn to see the features described and to form an idea of the relative standards which the maker of the key had in mind. To aid in this, the accompanying illustrations will be of advantage. When the reader becomes familiar with the key, the work of identifying any given piece will be comparatively easy. The material to be examined must, of course, be suitably prepared. It should be moistened; all cuts should be made with a very sharp knife or razor and be clean and smooth, for a bruised surface reveals but little structure. The most useful cut may be made along one of the edges. Instructive, thin, small sections may be made with a sharp penknife or razor, and when placed on a piece of thin glass, moistened and covered with another piece of glass, they may be examined by holding them toward the light.

Finding, on examination with the magnifier, that it contains pores, we know it is not coniferous or non-porous. Finding no pores collected in the

\* From researches of Miss E. Gerry. See *Jour. Agric. Research*, Vol. 1, p. 464.

† The remainder of this chapter is mainly the joint product of Dr. B. E. Fernow and Mr. Filibert Roth.

‡ Hough's *Wood Sections* will be found both helpful and pleasing. About one hundred and fifty species of American woods are now so prepared by Mr. Romeyn Hough, Lowville, N. Y.—J. B. J.



spring wood portion of the annual ring, but all scattered (diffused) through the ring, we turn at once to the class of "diffuse-porous woods." We now note the size and manner in which the pores are distributed through the ring. Finding them very small and neither conspicuously grouped, nor larger nor more abundant in the spring wood, we turn to the third group of this class. We now note the pith-rays, and finding them neither broad nor conspicuous, but difficult to distinguish even with the magnifier, we at once exclude the wood from the first two sections of this group and place it in the third, which is represented by only one kind, cottonwood. Finding the wood very soft, white, and on the longitudinal section with a silky luster, we are further assured that our determination is correct. We may now turn to the list of woods and obtain further information regarding the occurrence, qualities, and uses of the wood.

Sometimes our progress is not so easy; we may waver in what group or section to place the wood before us. In such cases we may try each of the doubtful roads until we reach a point where we find ourselves entirely wrong and then return and take up another line; or we may anticipate some of the later-mentioned features and, finding them apply to our specimen, gain additional assurance of the direction we ought to travel. Color will often help us to arrive at a speedy decision. In many cases, especially with conifers, which are rather difficult to distinguish, a knowledge of the locality from which the specimen comes is at once decisive. Thus, Northern white cedar, and bald cypress, and the cedar of the Pacific will be identified even without the somewhat indefinite criteria given in the key.

## KEY TO THE MORE IMPORTANT WOODS OF NORTH AMERICA

### I. NON-POROUS WOODS

Includes all coniferous woods.

#### A. Resin-ducts wanting.\*

##### 1. No distinct heartwood.

- a. Color effect yellowish white; summer wood darker yellowish (under microscope pith-ray without tracheids)..... **FIRS**
- b. Color effect reddish (roseate) (under microscope pith-ray with tracheids)..... **HEMLOCK**

##### 2. Heartwood present, color decidedly different in kind from sapwood.

- a. Heartwood light orange-red; sapwood pale lemon; wood heavy and hard..... **YEW**
- b. Heartwood purplish to brownish red; sapwood yellowish white; wood soft to medium hard, light, usually with aromatic odor.... **RED CEDAR**
- c. Heartwood maroon to terra cotta or deep brownish red; sapwood light

\* To discover the resin-ducts a very smooth surface is necessary, since resin-ducts are frequently seen only with difficulty, appearing on the cross-section as fine whiter or darker spots normally scattered singly, rarely in groups, usually in the summer wood of the annual ring. They are often much more easily seen on radial, and still more so on tangential sections, appearing there as fine lines or dots of open structure of different color, or as indentations or pin-scratches in a longitudinal direction.

orange to dark amber, very soft and light, no odor; pith-rays very distinct, specially pronounced on radial section. . . . . REDWOOD

**3. Heartwood present, color only different in shade from sapwood, dingy-yellowish brown.**

*a.* Odorless and tasteless. . . . . BALD CYPRESS

*b.* Wood with mild resinous odor, but tasteless. . . . . WHITE CEDAR

*c.* Wood with strong resinous odor and peppery taste when freshly cut.

INCENSE CEDAR

**B. Resin-ducts present.**

**1. No distinct heartwood; color white, resin-ducts very small, not numerous.**

SPRUCE

**2. Distinct heartwood present.**

*a.* Resin-ducts numerous, evenly scattered through the ring.

*a'.* Transition from spring wood to summer wood gradual; annual ring distinguished by a fine line of dense summer-wood cells; color white to yellowish red; wood soft and light. SOFT PINES \*

*b'.* Transition from spring wood to summer wood more or less abrupt; broad bands of dark-colored summer wood; color from light to deep orange; wood medium hard and heavy. . . . . HARD PINES \*

*b.* Resin-ducts not numerous nor evenly distributed.

*a'.* Color of heartwood orange-reddish; sapwood yellowish (same as hard pine); resin-ducts frequently combined in groups of 8 to 30, forming lines on the cross-section (tracheids with spirals).

DOUGLAS SPRUCE (DOUGLAS FIR)

*b'.* Color of heartwood light russet-brown; of sapwood yellowish brown; resin-ducts very few, irregularly scattered (tracheids without spirals). . . . . TAMARACK

ADDITIONAL NOTES FOR DISTINCTIONS IN THE NON-POROUS GROUP.

Spruce is hardly distinguishable from fir, except by the existence of the resin-ducts, and microscopically by the presence of tracheids in the medullary rays. Spruce may also be confounded with soft pine, except for the heartwood color of the latter and the larger, more frequent, and more readily visible resin-ducts.

In the lumber-yard hemlock is usually recognized by color and the slivery character of its surface. Western hemlocks partake of this last character to a less degree.

Microscopically the white pine can be distinguished by having usually only one large pit, while spruce shows three to five very small pits in the parenchyma-cells of the pith-ray communicating with the tracheid.

The distinction of the pines is possible only by microscopic examination. The following distinctive features may assist in recognizing, when in the log or lumber-pile, those usually found in the market:

The light, straw color, combined with great lightness and softness, distinguishes the white pines (white pine and sugar-pine) from the hard pines (all others in the market), which may also be recognized by the gradual change of spring wood into summer wood. This change in hard pines is abrupt, making the summer wood appear as a sharply defined and more or less broad band.

The Norway pine, which may be confounded with the shortleaf pine, can be distinguished by being much lighter and softer. It may also, but more rarely, be con-

\*Soft and hard pines are arbitrary distinctions, and the two are not distinguishable at the common limit.

## II. RING-POROUS WOODS

[Some of Group D and cedar-clm imperfectly ring-porous.]

A. Pores in summer wood minute, scattered singly or in groups, or in short broken lines, the course of which is never radial.

1. Pith-rays minute, scarcely distinct.

a. Wood heavy and hard; pores in the summer wood not in clusters.

a'. Color of radial section not yellow.....ASH

b'. Color of radial section light yellow; by which, together with its hardness and weight, this species is easily recognized.

OSAGE ORANGE

b. Wood light and soft; pores in the summer wood in clusters of 10 to 30.

CATALPA

founded with heavier white pine, but for the sharper definition of the annual ring, weight, and hardness.

The longleaf pine is strikingly heavy, hard, and resinous, and usually very regular and narrow-ringed, showing little sapwood, and differing in this respect from the shortleaf and loblolly pine, which usually have wider rings and more sapwood, the latter excelling in that respect.

If the pith is present in the cross-section, the following method, which was proposed by Mr. Arthur Koehler of the Forest Products Laboratory, serves to distinguish longleaf pine:

Make the pith and surrounding rings clearly visible by smoothing with a knife and moistening the smoothed surface. By the aid of a finely graduated rule and low-power lens measure the diameter of the pith. If it is less than 0.10 in. the specimen is not longleaf. In case the diameter of the pith is over 0.10 in., measure the diameter of the second annual ring, being careful to avoid mistake in identifying the second ring. On a piece of cross-sectional paper mark the diameter of pith as ordinates ( $y$ ) and the diameter of the second annual ring as abscissas ( $x$ ). Using scales of 1.0 in. = 0.05 for diameter of pith and 1.0 in. = 0.25 in. for diameter of second ring, draw a smooth curve through the following points:  $x=0.40, y=0.09$ ;  $x=0.75, y=0.12$ ;  $x=1.05, y=0.151$ ;  $x=1.50, y=0.198$ ;  $x=2.00, y=0.257$ . From the curve find diameter of pith corresponding to the measured diameter of the second ring. If this value is smaller than the measured diameter of pith, the specimen is longleaf, or very rarely Cuban pine. If the measured diameter of pith is nearly the same as the chart diameter, make check measurements on the other end of the specimen.

The following convenient and useful classification of pines into four groups, proposed by Dr. H. Mayr, is based on the appearance of the pith-ray as seen in a radial section of the spring wood of any ring:

Section I. Walls of the tracheids of the pith-ray with dentate projections.

a. One to two large, simple pits to each tracheid on the radial walls of the cells of the pith-ray.—Group 1. Represented in this country only by *P. resinosa*.

b. Three to six simple pits to each tracheid, on the walls of the cells of the pith-ray.—Group 2. *P. taeda, palustris*, etc., including most of our "hard" and "yellow" pines.

Section II. Walls of tracheids of pith-ray smooth, without dentate projections.

a. One or two large pits to each tracheid on the radial walls of each cell of the pith-ray.—Group 3. *P. strobus, lambertiana*, and other true white pines.

b. Three to six small pits on the radial walls of each cell of the pith-ray.—Group 4. *P. parryana* and other nut-pines, including also *P. balfouriana*.

2. Pith-rays very fine, yet distinct; pores in summer woods usually single or in short lines; color of heartwood reddish brown; of sapwood yellowish white; peculiar odor on fresh section.....**SASSAFRAS**
  3. Pith-rays fine, but distinct.
    - a. Very heavy and hard; heartwood yellowish brown.....**BLACK LOCUST**
    - b. Heavy; medium hard to hard.
      - a'. Pores in summer wood very minute, usually in small clusters of 3 to 8; heartwood light orange-brown.....**RED MULBERRY**
      - b'. Pores in summer wood small to minute, usually isolated; heartwood cherry-red.....**COFFEE-TREE**
  4. Pith-rays fine, but very conspicuous, even without magnifier. Color of heart wood red; of sapwood pale lemon.....**HONEY-LOCUST**
- B. Pores of summer wood minute or small, in concentric wavy and sometimes branching lines, appearing as finely feathered hatchings on tangential section.**
1. Pith-rays fine, but very distinct; color greenish white. Heartwood absent or imperfectly developed.....**HACKBERRY**
  2. Pith-rays indistinct; color of heartwood reddish brown; sapwood grayish to reddish white.....**ELMS**
- C. Pores of summer wood arranged in radial branching lines (when very crowded radial arrangement somewhat obscured).**
1. Pith-rays very minute, hardly visible.....**CHESTNUT**
  2. Pith-rays very broad and conspicuous.....**OAK**
- D. Pores of summer wood mostly but little smaller than those of the spring wood, isolated and scattered; very heavy and hard woods. The pores of the spring wood sometimes form but an imperfect zone. (Some diffuse-porous woods of groups A and B may seem to belong here.)**

## ADDITIONAL NOTES FOR DISTINCTIONS IN THE RING-POROUS GROUP

Sassafras and mulberry may be confounded but for the greater weight and hardness and the absence of odor in the mulberry; the radial section of mulberry also shows the pith-rays conspicuously.

Honey-locust, coffee-tree and black-locust are also very similar in appearance. The honey-locust stands out by the conspicuousness of the pith-rays, especially on radial sections, on account of their height, while the black locust is distinguished by the extremely great weight and hardness, together with its darker brown color.

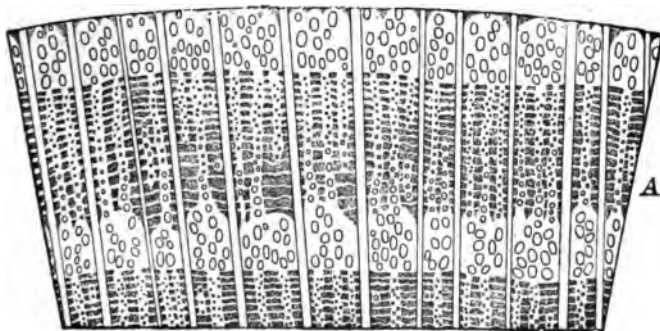


FIG. 19.—Wood of Coffee-tree.

1. Fine concentric lines \* (not of pores) as distinct, or nearly so, as the very fine pith-rays; outer summer wood with a tinge of red; heartwood light reddish brown. . . . . **HICKORY**
2. Fine concentric lines,\* much finer than the pith-rays; no reddish tinge in summer wood; sapwood white; heartwood blackish. . . . . **PERSIMMON**

The ashes, elms, hickories, and oaks may, on casual observation, appear to resemble one another on account of the pronounced zone of porous spring wood. The sharply defined large pith-rays of the oak exclude these at once; the wavy lines of pores in the summer wood, appearing as conspicuous, finely feathered hatchings on tangential section, distinguish the elms; while the ashes differ from the hickory by the very conspicuously defined zone of spring-wood pores, which in hickory appear more or less interrupted. The reddish hue of the hickory and the more or less brown hue of the ash may also aid in ready recognition. The smooth, radial surface of split hickory will readily separate it from the rest.

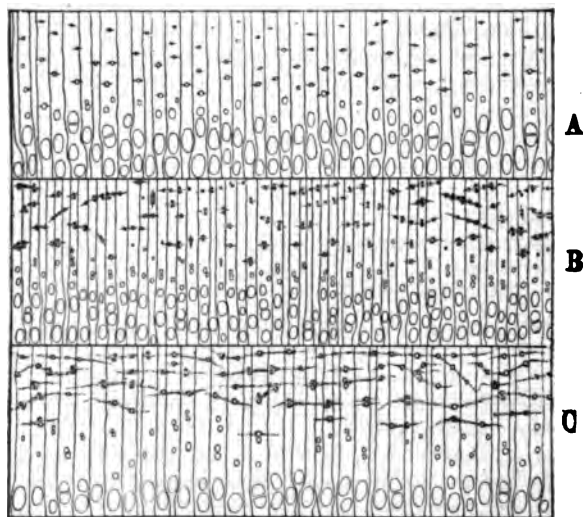


FIG. 20.—A, Black Ash; B, White Ash; C, Green Ash.

The different species of ash may be identified as follows:

1. Pores in the summer wood more or less united into lines by parenchyma fibers.
  - a. The lines short and broken, occurring mostly near the limit of the ring. **WHITE ASH**
  - b. The lines quite long and conspicuous in most parts of the summer wood. **GREEN ASH**
2. Pores in the summer wood not united into lines, or rarely so.
  - a. Heartwood reddish brown and very firm. . . . . **RED ASH**
  - b. Heartwood grayish brown and much more porous. . . . . **BLACK ASH**

In the oaks two groups can be readily distinguished by the manner in which the pores are distributed in the summer wood. In the white oaks the pores are very fine and numerous and crowded in the outer part of the summer wood, while in the black

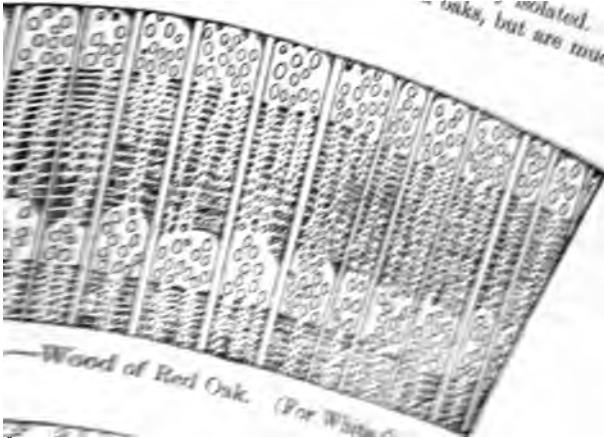
\* These fine concentric lines are the several ends of wood parenchyma fibers.—M. O W.

# KEY TO THE MORE IMPORTANT WOODS

## ADDITIONAL NOTES continued

the pores are larger, few in number, and mostly isolated. The live oaks, structure is concerned, belong to the black oaks, but are much less porous, being heavily and hard.

175



Wood of Red Oak. (For White Oak see p. 174.)

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SUGAR-MAPLE

## III. DIFFUSE-POROUS WOODS

[A few indistinctly ring-porous woods of Group II, D, and cedar-elm may seem to belong here.]

- A. Pores varying in size from large to minute; largest in spring wood, thereby giving sometimes the appearance of a ring-porous arrangement.
1. Heavy and hard; color of heartwood (especially on longitudinal section) chocolate-brown.....BLACK WALNUT
  2. Light and soft; color of heartwood light reddish brown.....BUTTERNUT
- B. Pores all minute and indistinct; most numerous in spring wood, giving rise to a lighter-colored zone or line (especially on longitudinal section), thereby appearing sometimes ring-porous; wood hard, heartwood vinous-reddish; pith-rays very fine, but very distinct. (See also the sometimes indistinct ring-porous cedar-elm, and occasionally winged elm, which are readily distinguished by the concentric wavy lines of pores in the summer wood.).....CHERRY
- C. Pores minute or indistinct, neither conspicuously larger nor more numerous in the spring wood and evenly distributed.
1. Broad pith-rays present.
    - a. All or most pith-rays broad, numerous, and crowded, especially on tangential sections, medium heavy and hard, difficult to split.  
SYCAMORE
    - b. Only part of the pith-rays broad.
      - a'. Broad pith-rays well defined, quite numerous; wood reddish white to reddish.....BEECH
      - b'. Broad pith-rays not sharply defined, made up of many small rays, not numerous. Stem furrowed, and therefore the periphery of section, and with it the annual rings, sinuous, bending in and out, and the large pith-rays generally limited to the furrows or concave portions. Wood white, not reddish.  
BLUE BEECH

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ADDITIONAL NOTES FOR DISTINCTIONS IN THE DIFFUSE-POROUS GROUP

Cherry and birch are sometimes confounded. The high pith-rays on the cherry or radial sections readily distinguish it; distinct pores on birch and spring-wood zone in cherry, as well as the darker vinous-brown color of the latter, will prove helpful.

Two groups of birches can be readily distinguishable, though specific distinction is not always possible.

1. Pith-rays fairly distinct, the pores rather few and not more abundant in the spring wood; wood heavy, usually darker.

CHERRY BIRCH and YELLOW BIRCH

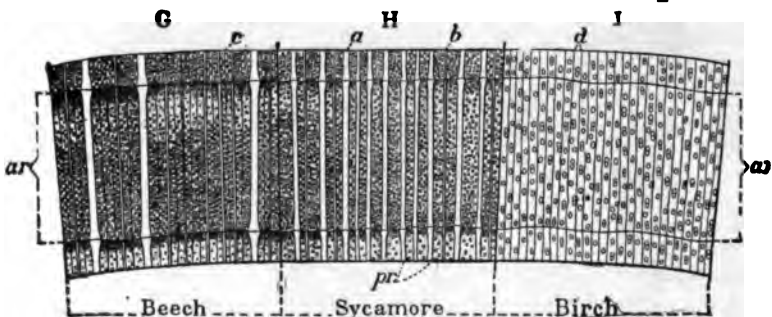


FIG. 24.—Wood of Beech, Sycamore, and Birch.

**2. No broad pith-rays present.****a. Pith-rays small to very small, but quite distinct.****a'. Wood hard.**

**a''.** Color reddish white, with dark reddish tinge in outer summer wood.....**MAPLE**

**b''.** Color white, without reddish tinge.....**HOLLY**

**b'. Wood soft to very soft.**

**a''.** Pores crowded, occupying nearly all the space between pith-rays.

**a'''.** Color yellowish white, often with a greenish tinge in heartwood.....**TULIP-POPLAR**  
**CUCUMBER-TREE**

**b'''.** Color of sapwood grayish, of heartwood light to dark reddish brown.....**SWEET GUM**

**b''.** Pores not crowded, occupying not over one-third the pith-rays; heartwood brownish white to very light brown

**BASSWOOD**

**b. Pith-rays scarcely distinct, yet if viewed with ordinary magnifier, plainly visible.****a'. Pores indistinct to the naked eye.**

**a''.** Color uniform pale yellow; pith-rays not conspicuous even on the radial section.....**BUCKEYE**

**b''.** Sapwood yellowish gray, heartwood grayish brown; pith-rays conspicuous on the radial section.....**SOUR GUM**

**b'.** Pores scarcely distinct, but mostly visible as grayish specks on the cross-section; sapwood whitish, heartwood reddish.

**BIRCH**

**3. Pith-rays not visible or else indistinct, even if viewed with magnifier.****1. Wood very soft, white, or in shades of brown, usually with a silky luster.**

**COTTONWOOD (POPLAR)**

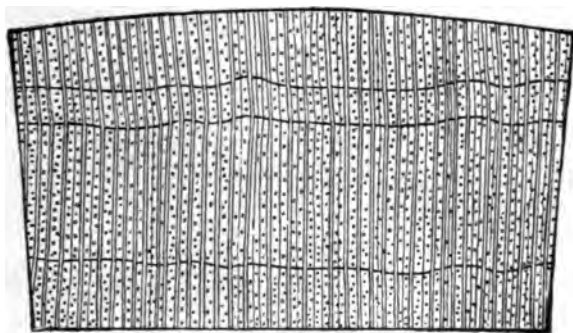
**2. Pith-rays barely distinct, pores more numerous and commonly forming a more porous spring wood zone; wood of medium weight.**

**CANOE OR PAPER-BIRCH**

The species of maple may be distinguished as follows:

**1. Most of the pith-rays broader than the pores and very conspicuous.**

**SUGAR-MAPLE**



**FIG. 25.—Wood of Maple.**



ADDITIONAL NOTES—*continued*

2. Pith-rays not or rarely broader than the pores, fine but conspicuous.
  - a. Wood heavy and hard, usually of darker reddish color and commonly spotted on cross-section.....RED MAPLE
  - b. Wood of medium weight and hardness, usually light-colored.

SILVER MAPLE

Red maple is not always safely distinguished from soft maple. In box-elder the pores are finer and more numerous than in soft maple.

The various species of elm may be distinguished as follows:

1. Pores of spring wood form a broad band of several rows; easy splitting, dark brown heart.....RED ELM
2. Pores of spring wood usually in a single row, or nearly so.
  - a. Pores of spring wood large, conspicuously so.....WHITE ELM
  - b. Pores of spring wood small to minute.
    - a'. Lines of pores in summer wood fine, not as wide as the intermediate spaces, giving rise to very compact grain....ROCK-ELM
    - b'. Lines of pores broad, commonly as wide as the intermediate spaces.

WINGED ELM

- c. Pores in spring wood indistinct, and therefore hardly a ring-porous wood.....CEDAR-ELM

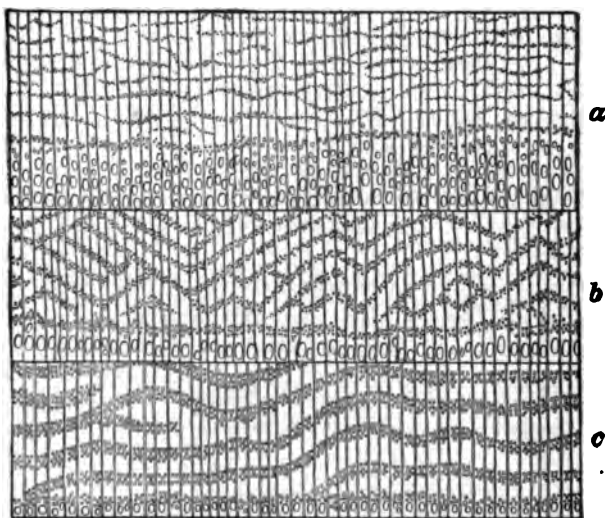


FIG. 26.—Wood of Elm. a, Red Elm; b, White Elm; c, Winged Elm.

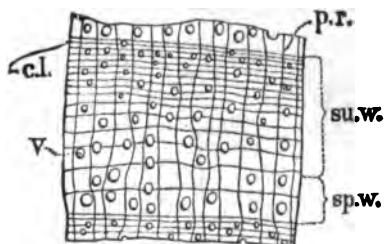


FIG. 27.—Walnut, p. r., pith-rays; c. l., concentric lines; v, vessels or pores; su. w, summer wood; sp. w, spring wood.

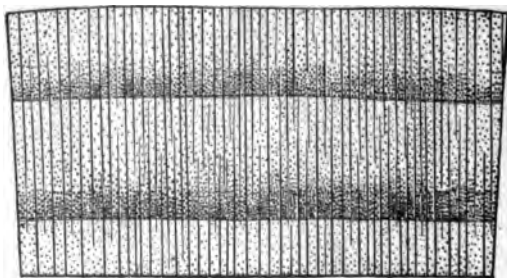


FIG. 28.—Wood of Cherry.

## CHAPTER V

### THE DETERIORATION AND PRESERVATION OF TIMBER \*

#### DETERIORATION

**201. The Durability of Wood.**—The durability of wood is a decidedly variable property. If well-seasoned and kept in a dry place, if immersed in water, or if buried in the ground, it often lasts for centuries. Examples of sound wood piling which have been buried over a thousand years, wooden buildings which have stood for centuries, and many wooden relics can be cited as proof of this statement. When, however, unprotected wood is subjected to moisture, air and moderate warmth it decays. The rapidity with which it decays depends on external conditions, the species of the wood, its preliminary conditioning, and its structure. Thus in mines the life of timber sets of untreated red oak and pine is not over two or three years, in ties or fence posts it may reach four to six years, and unprotected pine bleachers may last ten years. In exposed structures decay nearly always starts at the sills and bottoms of posts and columns. Joints like Fig. 1 afford receptacles for the collection of water and snow with the result shown. On the other hand untreated fence posts of osage orange,



FIG. 1.—Decay at Joint and in Strut Supporting a Bleachers. (Teesdale in *Am. Lumberman*, Oct. 3, 1914.)

\* *Preservation of Structural Timber*, by H. F. Weiss, *Bulletins* Nos. 78, 118, 107, and 126 of the Forest Service, were the principal sources for the compilation of this chapter.

black locust and red cedar often last a quarter century or more;\* the life of cedar poles may be estimated at fourteen years but those of loblolly pine are likely to decay in one-third of that time. In general sapwood decays much more rapidly than heartwood. Of the effects of conditioning and structure we shall say more presently.

Besides decay, wood may be injured by the attack of insects, marine borers, and woodpeckers. It deteriorates under mechanical abrasion and may, of course, be entirely destroyed by fire.

**202. Composition of Wood.**—Wood is essentially an organic substance consisting of over 99 per cent organic and under 1 per cent inorganic matter. It is made up of a skeleton of cellulose impregnated with lignin and the inorganic material composing the ashes. Cellulose is a whitish substance, like starch ( $C_6H_{10}O_5$ ), in composition, but more highly resistant to alcoholic fermentation. Lignin is also composed of carbohydrate compounds, but it is more soluble in acid than cellulose. Chemically, dry wood contains 49 per cent carbon, 44 per cent oxygen, 6 per cent hydrogen and 1 per cent ash; cellulose is made up of 44.4 per cent carbon, 49.4 per cent oxygen and 6.2 per cent hydrogen.

**203. Causes of Decay.**—The organic substances in wood are susceptible of attack both by bacteria and by fungi. Bacteria are very low forms of plant life, often only a single cell, which multiply by cell division and subdivision. Often a large number of cells unite to form minute filaments. The method by which bacteria decompose wood is not well understood, but it is probably similar in nature to a fungus attack. Fungi are also low forms of plant life which live through the destruction of other plants. They reproduce through thousands of minute particles, called "spores," which are blown about by the wind. Whenever one of these spores comes to rest it sends out microscopic filaments, "mycelia," which penetrate surrounding plant life in search of food. Sometimes, when the distance is not too great, mycelia from the fungus on a decaying timber is wafted across to a sound stick, thus communicating the disease. The destruction of the wood tissue is brought about by solvent chemicals "enzymes," which are secreted in the mycelia.

Only a small proportion of fungi destroy wood. Of this number some attack the lignin, others the cellulose, and still others consume both of these substances. The attack may proceed without any external evidence of the injury which the mycelia are inflicting within the wood, or it may be proclaimed by the appearance of mushroom growths, termed "fruiting bodies," on the surfaces of the timbers. In either case, after a considerable proportion of the cell walls has been destroyed by the mycelia, the wood becomes brittle and weak. Decaying timber is further characterized by a lack of resonance when struck with a hammer, by an abnormal

\* See *Bull.* No. 219, Ohio Agric. Expt. Sta.

capacity for absorbing water, and very often by an unnatural odor and color.

For life and propagation, fungi require air, moisture, warmth, and food supply. Not all fungi, however, thrive equally well under the same conditions. For example, the house fungus (*Merulius lachrymans*), Fig. 2 and 3, can live in air-seasoned timber surrounded by atmosphere with less than 70 per cent relative humidity, thrives at normal room temperatures, but is killed in an hour by a temperature of 115° F.\* This fungus has been known to lie dormant for several years in a seasoned stick of timber. All that is required to revive it is an increase in the humidity of its habitat. Frequently the house fungus furnishes no surface indication of its presence. In such cases boring into beams or planks which are thought to be contaminated may reveal the extent of the rotting. Rotted timber forms brownish chips. If the fungus is alive its presence may be detected by cutting small cubes from the edges of the brown wood region. These should be soaked in a 2 per cent citric acid solution for about six hours; they should then be removed and stored in a closed jar at 75° F. for a couple of weeks. If filaments appear the fungus is alive. The insidiousness of the attack of the house fungus makes it most dangerous, especially in buildings of mill construction type. On account of the virility of the fungus under somewhat dry conditions, the name "dry rot" has been given to this form of decay.

In contrast to the house fungus with its dry habitat and abnormal sensitiveness to heat, the following fungi which are characterized by many pores in their fruiting bodies may be mentioned: the *Fomes roseus*, *Trametes serialis* and the *Lenzites sepiaria*. The *Fomes roseus*, Fig. 4, has a hard, pink fruiting body covered with small round pores; it lives in a saturated atmosphere and works much mischief to wood exposed in damp



FIG. 2.—Strands of the House Fungus Found on Pine Planks at the Base of a Lumber Pile. (Photo by C. J. Humphrey.)

\*This discussion of the house fungus and dry rot is abstracted from a valuable article entitled "Dry Rot in Factory Timbers," by F. J. Hoxie, of the Inspection Dept. of Assoc. Factory Mutual Fire Ins. Co., Boston.

basements. The fruiting body of the *Trametes serialis*, Fig. 5, is tough; the surface is white and covered with small pores. It also works in a very moist atmosphere. The *Lenzites sepiaria* has a semi-circular plate-like



FIG. 3.—Portion of the Fruiting Body of *Merulius Lachrymans*. Taken from a Coniferous Timber. (Photo by C. J. Humphrey.)

fruiting body which has side attachment to the wood. The under side of its fruiting body is covered with gill-like pores. It is very active in destroying warehouse platforms and railroad ties. This fungus lives even when the temperature approaches the boiling point of water.



FIG. 4.—*Fomes Roseus* on the End of a Tie. (Photo by C. J. Humphrey.)

discoloration, which goes by the name of sap stain, is objectionable since it decreases the value of the timber.

Certain fungi attack with avidity the products stored in the cell walls of the sapwood. This attack is most common in woods which are air-seasoned in a warm humid atmosphere. Despite the fact that no great damage to mechanical properties \* appears to attend such action, at least in the early stages, yet the

\* See tests in Circular No. 192 of Forest Service.

**204. Insects.**—Although decay is the principal cause of deterioration of timber, an immense amount of danger is done annually by the attacks of insects. Timber with the bark on is especially liable to injury from them, and the attack once started in the green log may continue after the wood has been seasoned. Insects are particularly active in mine timbers, posts, poles, hickory hoops and poles, wagon stock, and pulpwood. Two common insects described by Weiss \* are the powder-post insect and the pole-borer. Both of these insects evolve from small beetles. The powder-post variety comes from a small brown or black bug which, when out of doors, deposits its eggs early in the spring on the surface of the wood. The eggs hatch into a small white grub. The grub bores into the wood and transforms it into a fine powder while selecting its food. After a period of growth the worm forms a cavity in the wood and lies dormant while its legs and wings are being developed. Since this beetle multiplies very rapidly, the deterioration of the infested timber proceeds quickly.



FIG. 5.—*Trametes Serialis*. Upper specimen from under side of a floor, lower specimen from side of a girder. (Photo by C. J. Humphrey. Reduced about one-half.)

The pole-borer comes from a reddish brown beetle about two-fifths to four-fifths of an inch long which deposits its eggs near the ground line of posts and poles during the late summer and early fall. On hatching, the creamy white grub bores into the wood. It transforms the wood tissue into a reddish brown or yellow dust which is packed into the burrow behind the worm. Like the powder-post insect, the pole-borer lies dormant in a cell in the wood until converted into a beetle. During the late summer the latter emerges from the pole through a large hole near the ground line. The pole-borer attacks both sound and decayed timber but is not active in the latter if it is water-soaked. It has been found in poles within two years after setting in the ground.

**205. Marine Borers.**—There are two classes of marine borers infesting the waters of both Atlantic and Pacific Coasts, the mollusk and the crustacean types. Of the first class the teredo and xylotrya, which are very similar in appearance and mode of living, are the most important. Owing

\* See *The Preservation of Structural Timber*, Ch. II.

to their shape they are frequently called "shipworms." Either mollusk tunnels into the wood by means of a pair of shell valves and excretes the borings and a calcareous substance for lining the burrow through a posterior syphon. The food supply of the mollusk which consists of low forms of animal life found in the water, is secured through a second posterior syphon. The teredo rarely exceeds a length of 15 in. or a diameter of  $\frac{3}{8}$  in., but specimens of *xylotrya* 6 ft. long by 1 in. in diameter have been reported. Shipworms infest warm salt water or brackish waters and are said to prefer calcareous shores. They attack piling between mean tide and the low water level; and at the Isthmus of Panama and along the coast of Florida have been known to ruin untreated timber in less than one year.

The limnoria or wood louse is the crustacean which is most dangerous to timber. It grows to the size of a grain of rice and bores into the wood by means of sharp jaws. It lives on food gotten from the wood substance which it penetrates radially to a depth of about  $\frac{1}{4}$  in. per year. It is active only in clear salt water and confines its attack to a narrow belt around the piling near the low-water mark. The limnoria is particularly active in the Gulf of Mexico and along the north Pacific Coast.

No native timber except the palmetto appears to be highly resistant to the attack of marine borers. The greenheart of South America and the jarrah of Australia are also said to be highly resistant to such attack. Pine and fir are the timbers largely used for piling in this country, but they must be protected to withstand the ravages of these pests. Creosoting by the boiling or Bethell processes, or encasing the piling in concrete jackets are the methods of protection ordinarily used. Impregnation with creosote even when well done is not always proof against shipworms, while concrete casings are expensive and likely to be cracked. Borers in an attacked pile may be killed by chlorine gas, which is generated as follows: The pile is enclosed with a canvas curtain and an electric current passed through the pile and the enclosed salt water. The treatment is expensive and requires frequent repetitions in waters which are heavily infested with borers.

**206. Other Deteriorating Influences.**—Wood cross-ties, mine props and wharf timbers suffer considerable from mechanical wear. It is estimated that a tenth of the annual tie loss could be saved by the use of suitable tie-plates and improved spikes. It is not always possible or economical to protect timbers from mechanical wear, but in some cases iron plates may be effectively employed as shields.

Fire decomposes wood into carbon dioxide, water vapor and ash. Wet wood is about twice as resistant to fire as dry wood. Structural timbers which are well-seasoned will ignite with difficulty at temperatures in the vicinity of 400° F., and very quickly at temperatures around 600° F. Wood attacked by dry rot is more combustible than sound timber. Experi-

ments by the National Fire Protection Association\* show that the resistance of wood to fire can be much increased by saturating it with weak solutions (5 to 10 per cent) of ammonium sulphate or ammonium phosphate. Impregnation of the chemicals was accomplished by heating the solutions to 150° F. and maintaining a pressure of 130 lb. per square inch on the specimens for two hours while they were soaking. For wood exposed to moisture a treatment with zinc borate is effective. In this treatment the wood is first impregnated with a 10 per cent borax solution; it is then dried and again soaked in a 3 per cent zinc chloride solution. The two compounds react forming the insoluble borate which remains in the wood. The estimated cost of these treatments per thousand shingles ranged from \$1.29 for the zinc borate treatment to \$2.48 for a treatment with a solution of 4 per cent ammonium sulphate and 3 per cent ammonium phosphate. In view of the enormous annual fire loss in timber constructions, further experimentation along these lines should be done.

Woodpeckers do considerable injury to poles by boring into them and building their nests. Where the holes are well above the ground line, they cause little direct loss in strength or stiffness, but they afford excellent breeding grounds for fungi and thus may foster decay.

#### PRESERVATION

**207. The Need of Preservation.**—Statistics compiled by the Forest Service † show that the average life in years of untreated structural timbers in the United States is approximately as follows: Mine props, 3; piles, 3½; ties, 7; posts, 8; lumber subject to decay, 8; poles, 13; and shingles, 18. Although statistics are not given, it is probable that between seventeen and twenty billion board feet of structural timber are used annually for replacements. Weiss estimates that the amount of timber cut for such purposes could be decreased annually by nearly seven billion board feet, if proper preservative methods were practiced. This would effect a net saving of not far from a hundred million dollars a year. From a consideration of the low durability of wood and the great cost of the quantity required for replacements, the need of practicing comparatively inexpensive methods of preservation becomes evident. Furthermore, the proper use of efficient protectives would lead to the planting of more rapidly growing trees, the more effective utilization of inferior trees and top logs, the clearing of land occupied by fire-killed timber (since the latter can be effectively used if treated); in short preservation would lead to better forest management in general.

At the present time (1916) about one-third of the cross-ties and a like

\* Rept. of Common Uses of Wood in *Proceedings* of 1915.

† See *Bulletin* 78.



proportion of the piling annually used in the United States is given a preservative treatment. The proportion of building and bridge timbers which are so treated is, however, quite small. The entire amount of timber treated in this country is approximately a hundred and fifty million cubic feet per year. To treat this amount of wood about a hundred million gallons of creosote, about thirty-five million pounds of zinc chloride and nearly five million gallons of other preservatives are consumed.

**208. The Relations of Structure to the Penetrance of Preservatives.** Structure plays a very important role in determining the ease with which preservatives may be forced into wood and also in fixing the quantity injected. In most woods the sapwood is more easily impregnated and absorbs more preservative than the heartwood. However, in hemlock, alpine fir, and white spruce the sapwood is scarcely less resistant to penetration than the heartwood. Bark is nearly impenetrable and should always be completely removed from timber which is to be treated. The comparative resistance of the heart and sapwood should be considered in forming timbers which are to be treated. In the diffuse-porous woods and in those conifers which show little demarkation between spring and summer wood, the absorption of preservative is more uniform than in the ring-porous hardwoods or the hard pines. In the ring-porous woods most of the preservative will run into the spring wood, whereas in longleaf pine the greater part will be found in the dense summer wood. Owing to the difference in the absorption of preservative by the spring and summer wood of the hard pines, a minimum limit on the number of rings per inch is often placed in specifications. The purpose of this restriction on rate of growth is to prevent wide variations in the distribution of the preservative.

Within a given species it is likely that the absorption of preservative varies inversely as the density of the wood, but no such relation exists between timbers of different species. Thus, red oak and hard pine, which are comparatively heavy woods, absorb much more preservative than the light white spruce.

Since nearly all of the preservative is held in the cell cavities and only a small proportion permeates the cell walls, it follows that any condition which causes a plugging of these cells will interfere with the injection of preservatives. Such conditions are effected in many woods by tyloses (Art. 197, 198 and 199). If such woods are treated the preservative is likely to be very non-uniformly distributed.

In coniferous wood the resin-ducts, if unclogged by resin or growths, serve as canals for the rapid passage of preservatives. It is probable that the great absorption of the dense summer wood of the hard pines is due to the fact that it contains these ducts. Nearly all of the pines also possess radial resin-ducts which materially assist in the radial penetration

of preservatives. Radial ducts are lacking in the larches, hemlocks, firs and spruces; and it is much more difficult to secure a deep radial penetration in them than in the pines.

In some woods it is probable that the radial transmission of preservatives is affected through pit membranes in the cell walls. Tiemann and Weiss claim that slits which are opened in the cell walls during seasoning are another possible avenue of transmission.

Besides these physical characteristics of the wood structure, the chemical composition of the cell walls probably has an important influence on the absorption of preservatives.

Among the woods which are most readily injected with preservatives are longleaf pine, shortleaf pine, Western yellow pine, lodgepole pine, loblolly pine, red birch, white elm, red elm, soft maple, beech and red oak. White oak, alpine fir, Douglas fir, tamarack and white spruce are treated with difficulty. In this connection it should be recognized that the form of the timber often plays an important part in determining the penetration of preservation. For example, a Douglas fir pole can be easily impregnated with preservative because of its sapwood envelope. On the other hand, a large dimension timber of this species having considerable exposed heartwood would be treated with much difficulty on account of the resistance of the heartwood to penetration.

**209. The Treatment of Timber before Preservation.**—It is generally best to cut timber in the winter in order to avoid the attack of fungus and insects. Also for timber which is likely to check from rapid seasoning the best cutting time is in the fall or winter.

Soon after the tree has been felled every bit of the bark should be removed. By so doing danger of attack by fungi and insects is lessened, the weight of the wood is diminished and the surface of the wood is rendered much more permeable to preservatives. If, however, the wood is allowed to season too rapidly after the removal of the bark, it may become case-hardened and its resistance to penetration of liquids may be greatly increased.

It is desirable that all timber which is to be treated be thoroughly seasoned in order that the penetration of preservatives may be facilitated. With the exception of air-seasoning, exposure to saturated steam is the most-used conditioning process for timber which is to be preserved. Under this process the timber is placed in a large treating cylinder and subjected to live steam at a pressure of 20 to 40 lb. per square inch for two to ten hours, the time being dependent on the size and character of the timber. A vacuum of about 25 in. is then applied for thirty minutes to two hours, after which the timber is treated with preservative. This process is a preliminary stage in several of the common methods of treatment. With a few creosoting processes, seasoning is accomplished by running the timber

into a cylinder and soaking it in creosote. The oil is gradually heated until the temperature is raised above the boiling-point of water. This causes the latter to vaporize. The vapor is drawn off and condensed to free it of oil. After the moisture in wood has been sufficiently reduced, the cylinder is filled full of preservative and impregnation is begun.

#### SUPERFICIAL TREATMENTS

**210. Conditions for Use of Superficial Treatments.**—There are three inexpensive methods of treating the surface of timber to protect it against decay and insects. These methods are of value when the amount of timber to be treated is too small to warrant the erection of a treating plant; when it is impracticable to haul the timber to the work from a plant; or when it is necessary to do the work at a minimum cost. Since the value of every one of them is based upon the maintenance of an unbroken film which will resist the attack of fungi, it is very necessary that the timber shall be thoroughly air-seasoned before treating. If the timber is only partially seasoned or green when treated, it is likely to check subsequently and thus produce passage ways for insects and mycelia.

**211. Brush Treatments.**—Probably the most-used superficial treatments are those in which a liquid is applied to the surface of the timber by means of a brush. Creosote, paint, oil, and whitewash are among the liquids used for the purpose. Creosote should be heated to about 200° F. before applying to the wood, since heating considerably decreases the viscosity of the oil and thereby aids in securing penetration of the preservative. Great pains should be taken to thoroughly coat all defects and fill checks, shakes and joints. This method of treatment has been used considerably for the preservation of mine timbers, poles and posts. It is well adapted to use on farms.

**212. Dipping.**—By dipping the timber into the preservative and allowing it to scak for a few minutes, it is possible to secure a more complete coating of the defects than is gotten by brush treatments. The process requires the use of a large tank for holding the timber and necessitates a somewhat greater use of preservative, but the labor cost is less than in the brush process. It can be very effectively used for butt treatments on fence posts and poles. When used for this purpose the preservative should cover the pole for at least a foot above the ground line.

**213. Charring.**—A very old and inexpensive method for protecting wood consists in charring the outer fibers of the timbers by fire. This process produces an envelope of charcoal which, being devoid of food elements, is not attacked by fungi. If the strength of the pieces treated is of great importance this process is detrimental, because it destroys the outer fibers and injures those immediately beneath. It has been used for treating the butts of posts and poles, but is not very efficacious.

## NON-PRESSURE PROCESSES OF IMPREGNATION

**214. The Value of Non-pressure Processes.**—In the non-pressure processes the preservative is drawn into the wood by absorption or it is forced in by atmospheric pressure. By these processes it is not possible to secure as uniform and deep penetrations as with the pressure methods, but with woods like loblolly pine, shortleaf pine, red cedar and beech, they can be successfully used. These processes require a longer time for treatment than the pressure processes. On the other hand, since they use no heavy treating cylinder with its expensive equipment, they afford cheap and effective means of preserving small quantities of poles, mine timbers and ties, provided the wood is easily impregnated. The method is also of value when salts which would attack iron treating cylinders are used.

**215. Open-tank Process.**—In this treatment the timber is placed in a tank and covered with the preservative. The charge is then heated to a temperature just above the boiling point of water. This serves to expel a considerable proportion of air and moisture from the cells in the wood. After soaking at this temperature for an hour or two the timber may be allowed to cool with the liquid, or it may be transferred to a cold tank where it is kept for another hour or more depending on how deep a penetration is wanted. As the wood cools a vacuum is formed in the outer cells and the preservative is injected by atmospheric pressure. Sometimes, when a deep penetration is desired with a minimum expenditure of preservative, the timber is drawn from the cooling tank before it has entirely cooled. As the interior of the stick gradually cools vacuums are formed which are filled by the excess fluid held in the outer cells.

The process may be used with creosote, zinc chloride, or crude oil. However, if the wood is boiled in the zinc chloride solution, its strength is likely to be impaired.

**216. Kyanizing.**—In this process the timber, which must be thoroughly seasoned, is immersed in a 1 per cent solution of bichloride of mercury for a number of days. The time of treatment is dependent upon the thickness of the pieces and the depth of penetration desired. Ordinarily the time in days is equal to the thickness of the timber in inches plus one, and the depth of penetration does not exceed a quarter of an inch. Owing to the poisonous character of the salt great care must be exercised during the treatment of the timber; and the treated lumber should not be used where it is likely to be licked by animals.

As a means of preserving timber used in dry locations, this process ranks high; but on account of the poisonous nature of the salt, the long-time required for the process, and the solubility of the salt in water, it has not been widely used in this country.

## PRESSURE PROCESSES OF IMPREGNATION

**217. Field of Use.**—In general the pressure processes are the most satisfactory methods of treating large amounts of timber. Furthermore, these methods are the only ones which can be successfully used to impregnate many kinds of wood, such as hemlock, Douglas fir and redwood.

**218. Bethell or Full-cell Process.**—The timber for treatment is placed on small cars and carried into horizontal steel cylinders which are ordinarily about 130 ft. long by 7 ft. in diameter. Green timber is given a seasoning by the saturated steam process; this is unnecessary if the wood is well-seasoned. Coal-tar creosote oil is then admitted to the cylinder and forced into the timber by a pressure of 100 to 180 lb. per square inch. The pressure is maintained until the oil gages show the required impregnation. Then the oil is blown out of the cylinder into reservoirs, and the timber, after dripping for a few moments, is removed. In some plants dripping is accelerated by drawing a vacuum just before removing the timber. The process is much used both in this country and abroad. It is especially valuable for wood-block and piling where a heavy impregnation of oil is imperative. On account of the large expenditure of oil, 10 to 20 lb. per cubic foot, the process is very costly. The cost of treating cross-ties by it ordinarily runs between 40 and 50 cents each.

**219. Burnettizing.**—This process is performed in the same manner as the Bethell process, but differs in the preservative. An aqueous solution containing from 2 to 5 per cent of zinc chloride is used and about  $\frac{1}{2}$  lb. of salt per cubic foot is the average impregnation in the process. Owing to the solubility of zinc chloride in water, the process is not suited to treating timbers which are to be placed in damp locations. It is a very inexpensive process costing about one-third as much as the full-cell treatment. Burnettizing has been successfully used to treat ties and lumber both in the United States and in Europe.

**220. The Boiling Process.**—In this process either green or seasoned timber is given a conditioning treatment in creosote oil (see Art. 209) before impregnation with creosote. After the oil conditioning the remainder of the process is much like the Bethell method. The boiling method is used principally in preserving Douglas fir. Tests indicate that it may injure the strength of this wood. (Art. 243.)

**221. The Rueping Process.**—One of the most important processes both here and abroad is the Rueping empty-cell process with creosote. Air-seasoned timber is preferred, although steam treated material can be used. After placement in the treating cylinder, the cells of the wood are filled with compressed air which is admitted under a pressure of about 75 lb. per square inch. Oil is then admitted at a slightly higher pressure until the wood has been immersed, when the pressure is raised to 150 lb.

per square inch or more. After the proper amount of oil has been injected, the creosote is forced out of the cylinder and the pressure released. As the pressure is withdrawn the compressed air within the wood expels the excess oil into the cylinder. It is thus possible to secure a deep penetration and to coat the cell walls with 5 to 7 lb. of oil per cubic foot of wood. Treatments by this process cost about two-thirds as much as the full-cell methods.

**222. The Lowry Process.**—This process, like the Rueping, is planned to secure a deep penetration of oil with a small absorption. Air-dry timber is run into the cylinder and submerged in creosote oil at a temperature of about 200° F. Pressure is then applied, and the temperature and pressure are regulated until the timber has been filled with oil. After withdrawal of the oil a vacuum is drawn until the surplus oil in the wood cells has been removed. When the excess oil has been removed from the cylinder, the timber is taken out. The process is somewhat more expensive than the Rueping process but less costly than the full-cell. It is considerably used in the United States for treating cross-ties.

**223. The Card Process.**—Owing to the high cost of creosote oil and the solubility of zinc chloride neither of these preservatives has been universally adopted. The aim of the Card process is to lessen these objectionable features by using a mixture of 15 to 20 per cent of creosote with a 3 to 5 per cent solution of zinc chloride. In operating the process, air-seasoned timber is run into the treating cylinder and given a vacuum treatment for about an hour. It is next immersed in the preservative at a temperature of about 180° F. A pressure of approximately 125 lb. per square inch is then applied for several hours. During this period the mixture of oil and zinc chloride is stirred continuously by a centrifugal pump to prevent separation of the components. After impregnation has been finished, the preservative is drained from the cylinder, and a vacuum drawn to remove surplus preservatives from the wood cells. By the Card method approximately as much zinc chloride is injected as in Burnettizing together with about 3 to 5 lb. of creosote per cubic foot. The process costs approximately half as much as the Bethell process and has found favor in this country for the treatment of cross-ties.

#### PRESERVATIVES AND THE EFFICIENCY OF PRESERVATION

**224. Preservatives.**—Inasmuch as fungi cannot thrive without moisture, waterproofing of seasoned wood will render it resistant to attack. Crude oil, paint and stains are the common preservatives of this class. A far surer treatment is effected, however, by preservatives which poison the food supply of fungi and insects. The creosote oils and the inorganic salts,—zinc chloride, mercuric chloride and copper sulphate,—are members of the latter class.

The term crude oil includes three classes, (1) oil with a paraffin base,

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plus a balance which, with interest compounded annually, will provide a fund at the end of  $n$  years equal to the first cost. The estimates show the value of the durable woods and emphasize the importance of treating the inexpensive timber which is not naturally durable.

TABLE 1.—ESTIMATED ANNUAL SAVING DUE TO PRESERVATIVE TREATMENT OF CROSS-TIES

[A charge of 45 cents for tie-plates and placement was added to the cost of each tie before computing the annual charge. The estimated cost of creosoting was figured at 45 cents and of Burnettizing at 15 cents per tie.]

Kind of Tie.	ESTIMATED LIFE IN YR.			COST OF TIES IN DOLLARS.			ANNUAL CHARGE IN TRACK.			ANNUAL SAVINGS.	
	Untreated.	10 lb. creosote per cu. ft.	$\frac{1}{2}$ lb. $ZnCl_2$ per cu. ft.	Untreated.	10 lb. creosote per cu. ft.	$\frac{1}{2}$ lb. $ZnCl_2$ per cu. ft.	Untreated.	10 lb. creosote per cu. ft.	$\frac{1}{2}$ lb. $ZnCl_2$ per cu. ft.	10 lb. creosote per cu. ft.	$\frac{1}{2}$ lb. $ZnCl_2$ per cu. ft.
Redwood.....	12	...	...	0.60	...	...	0.117				
Cedar.....	11	...	...	0.55	...	...	0.120				
Cypress.....	10	...	...	0.55	...	...	0.128				
White oak.....	8	...	...	0.80	...	...	0.194				
Longleaf pine....	7	20	11	0.60	1.05	0.75	0.181	0.120	0.144	0.061	0.037
Douglas fir.....	7	15	11	0.50	.95	.65	0.164	0.135	0.132	0.029	0.032
Western larch....	7	15	11	0.40	.85	.55	0.147	0.124	0.120	0.023	0.027
Tamarack.....	5	15	11	0.50	.95	.65	0.219	0.135	0.132	0.084	0.087
Hemlock.....	5	15	11	0.40	.85	.55	0.197	0.124	0.120	0.073	0.077
Chestnut.....	5	12	9	0.50	.95	.65	0.219	0.157	0.155	0.062	0.064
Red oak.....	4	20	12	0.65	1.10	.80	0.310	0.124	0.140	0.186	0.170
Beech.....	4	20	12	0.50	.95	.65	0.268	0.112	0.123	0.156	0.145
Birch.....	4	20	12	0.50	.95	.65	0.268	0.112	0.123	0.156	0.145
Maple.....	4	18	12	0.50	.95	.65	0.268	0.119	0.123	0.149	0.145
Lodgepole pine...	4	15	11	0.65	1.10	.80	0.310	0.149	0.150	0.161	0.160
Western pine....	4	15	11	0.50	.95	.65	0.268	0.135	0.132	0.133	0.136
Loblolly pine....	3	15	10	0.40	.85	.55	0.312	0.124	0.129	0.188	0.183
Red gum.....	3	16	11	0.55	1.00	.70	0.367	0.134	0.138	0.233	0.229

## CHAPTER VI

### THE MECHANICAL PROPERTIES OF TIMBER

duction.—In order that the engineer may properly design beams for various parts of wooden structures, he must be conversant with the strength and stiffness of the available timber. He must also know how various defects and conditions affect the properties. The architect must not only appreciate the properties of the various species, the relative ease with which each may be made to shrink, warp and check; but he must likewise be able to proportion joists and rafters to carry the imposed loads without excessive deflection. The wheelwright must understand how the strength of his axles, spokes, and shafts are influenced by the rate of growth, density, and defects. The carpenter and the joiner must also have knowledge of the mechanical properties of wood so that they may work it to best advantage.

Moreover, wood of a given species is extremely variable. Trees vary widely in their rate of growth, due to climatic conditions, the character of the surrounding forest, the character of the soil and the physical region in which they grow. These conditions of growth, the age of the tree, the amount of moisture and the defects all influence the mechanical properties of a piece of timber of a given species.

Regarding the importance of a knowledge of the properties of wood and the factors influencing them, Dr. B. E. Fernow, chief of the Forestry Division of the U. S. Agricultural Department, inaugurated a series of investigations in 1891. In the beginning, the microscopic and physical tests were conducted at Washington and the mechanical tests were made on, at Washington University, St. Louis. Later the mechanical tests were distributed among a number of technical laboratories in various parts of the country. In 1909 the major portion of the work done in the Forest Products Laboratories was transferred to the Forest Products Laboratory, Madison, Wis.

Most of the tests of timber are only a portion of the work carried on by the Forest Products Laboratory. Studies of the physical and chemical properties, the structure, the methods of preservation of wood, and the utilization of wood products are among the other more important lines of investigation conducted by this institution.

The Forest Products Laboratory is run by the U. S. Government; buildings, light, heat, and maintenance are provided by The University of Wisconsin.



On account of the broad scope of these investigations, the great pains which are taken to identify species, to select properly both specimens and trees, and on account of the care taken to eliminate variables in testing, the results obtained are of very great value. Most of the data in this chapter is taken from publications of this laboratory.

The mechanical tests which are most commonly made at the Forest Products Laboratory are: compression, shear, both static and impact bending, tension, hardness, and cleavage. Observations are also made on the number of rings per inch, per cent summer wood, per cent moisture, density and shrinkage. Many of the more important conditions pertaining to the methods of testing wood have been considered in Ch. III. Supplementary information will be given, when necessary, in the discussions on mechanical properties which follow.

### THE STRENGTH OF WOOD

**227. Compressive Strength.**—When wood is subjected to compressive forces acting parallel to the axis of growth (parallel to the grain), it is, in proportion to its weight, one of the strongest of structural materials. Columns and posts are, therefore, often fashioned of it. Inasmuch as the strength of such a member is a function of the compressive strength and the slenderness ratio, information concerning the former is of much importance. Furthermore, a knowledge of the compressive strength is of value in estimating strength in bending, since experiments have demonstrated that the yield point of a wooden beam is determined by the compressive strength of the wood. The compression test is not, however, as effective in demonstrating the weakening influences of defects as the cross-bending test.

When wood is subjected to compression parallel to the grain it may fail through collapsing of the cell walls or through lateral bending of the cells and fibers. In most of the conifers where the cells (tracheids) have thin walls, failure begins at pits in the walls of one of these cells and gradually causes a collapse of the entire cell. The plane of rupture is generally inclined about  $60^{\circ}$  to  $75^{\circ}$  with the axis of the cell. From the inclination of the plane of failure it appears probable that final collapse is due to the weakness of the cell wall in transverse shear. Adjacent cells are overstressed by the failure of the individual cell and a wrinkling of the surface, showing the progress of the breakdown throughout the specimen, becomes visible. In wet wood and in the hardwoods, which are composed of thick-walled fibers and vessels, incipient failure is due to bending of the individual fibers. Occasionally after the wrinkling begins, the specimen is separated into groups of fibers by longitudinal cracks. This condition is brought about by splitting of the fibers and not by failure of the bond

# —RESULT TESTS

[Test spec

C GRAVITY. EN DRY, IED ON—		SSION PARALLEL TO GRAIN.		Compres- sion Per- pendicular to Grain, Fiber Stress at Elastic Limit.	HARDNESS—LOAD REQUIRED TO EMBED A 0.444-INCH BALL TO ONE-HALF ITS DIAMETER.			SHEARING STRENGTH PARALLEL TO GRAIN.		CLEAVAGE STR PER INCH OF 1	
ne a.	Volume When Oven Dry.	Crushing Strength.	Modulus of Elasticity.		End Surface.	Radial Surface.	Tangential Surface.	When Surface of Fall			
								Radial.	Tangential.	Radial.	Tan
		Lb. per Sq. in.	1000 Lb. per Sq. in.	[Lb. per Sq. in.	Pounds.	Pounds.	Pounds.	Lb. per Sq. in.	Lb. per Sq. in.	Pounds.	Po
	0.640	2200	422	422	665	542	546	876	832	275	
	.590	4220	1531	889	1121	1000	1017	1360	1312	333	
	.574	4040	1497	801	842	729	734	1239	1166	357	
	.669	1820	1016	197	273	226	217	560	617	130	
	.661	3480	1412	605	1012	897	918	1154	1375	339	
		3310	1915	439	827	768	739	1103	1188	294	
		3740		696	954	883	893	1210	1330	282	
	.639	3990	1453	730	919	757	887	1197	1174	401	
		2700		292	536	456	497	778	872	210	
		2690									
	.576	3310	1068	575	829	795	773	1095	1161	422	4
		4520	1658	994				1134	1191		
		3260	1207	1000				1094	1330		
		4570	1616	986				1134	1348		
		4600	1681	1065				1165	1375		
		4320	1359	958				1251	1313		
		3980	1411	938				1010	1053		
		4870	1474	1101				1200	1215		
		4760	1681	1224				1235	1020		
		4820	1911	1130				1312	1405		
		4820	1980	1114				1334	1457		
		5060	1869	1158				1205	1318		
		4360	1478	1080				1337	1505		
		4370	1029						1245		
		4600	1943	972				1230	1297		
		4660	1926	1088				1390	1490		
	.759	4970	1536	1684	1862	1860	1832	1885	2096	552	6
		3680		606	766	621	625	1130	1330	297	3
	.643	3670	1463	653	992	918	901	1193	1455	376	2
		4000		870	965	815	815	1340	1630	357	4
		3330	1062	1148	1139	1068	1081	1196	1402	354	4
	.732	3440	1335	844	1139	1044	1039	1215	1226	307	4
	.662	3210	1295	807	1107	1020	1002	1132	1195	380	4
	.660	3460	1924	682	924	884	901	1047	1059	365	4
		4360	1489	943	1205	1217	1099	1198	1304	428	5
	.792	4840		1355							
		3520		829	1113	1030	943	1096	1292	382	4
	.704	3530	1308	727	1087	1049	1047	1187	1425	342	4
	.695	3490	1181	1004	1183	1163	1147	1138	1308	371	4
	.708	3700	1465	912	1093	1083	1031	1162	1196	379	4
	.669	3080		802	847	800	790	1270	1320	337	3
	.838	5810	1331	2260	1838	2312	1762				
	.526	2790	1073	433	664	590	599	900	1102	265	4
	.545	3550	1280	451	814	666	733	978	1084	277	3
	.315	1990	754	288	321	225	228	617	614	148	1
		3030	868	518				613	662		
	.513	3960	1738	548	400	355	354	836	800	167	1
	.321	2060	882	307	284	203	235	573	654	130	1
		3040	1579	334				517	639		
	.458	2920	1440	427	415	399	416	853	858	139	1
	.437	2800	1332	445	381	322	334	742	723	145	1
	.394	2750	1054	420	463	354	334	790	813	168	1
	.415	2530	1219	364	288	307	316	672	747	142	1
	.407	2400	991	332	316	318	319	709	719	253	2
	.599	4280	1890	491	574	502	521	1060	953	187	1
	.507	3080	1646	358	355	345	340	812	741	161	1
		3570		400				746	670		
	.386	2600	1029	353	334	307	342	702	714	168	1
	.395	2220	1046	342	310	311	318	644	680	152	1
	.433	2420	1271	326	316	306	323	686	706	162	1
	.435	2600	1174	410	315	330	332	694	717	152	1
	.391	2720	1318	314	304	294	299	649	639	144	1
		3570	1175	539							
		4160	1272	618							
	.359	2170	1021	302	272	253	274	607	624	110	1
	.335	1800	971	279	231	216	226	538	600	129	1
		2920		322				754	793		
		1940		262				645	615		
	.558	3480	1566	480	401	380	370	883	843	167	1



between fibers.\* It will be generally noted that the line of failure on tangential surfaces of the specimen is inclined as previously stated; but on radial surfaces it is approximately normal to the axis of the test-piece. The direction in the latter case is much influenced by the medullary rays. In cross-grained pieces the failure is likely to take place through shear parallel to the grain.

The strength of timber compressed across the grain is brought into play wherever a concentrated load is imposed on a beam. Since the compressive strength across the grain is only a small fraction of the compressive strength parallel to the grain, proper allowance for this discrepancy must be made in designing columns resting on wooden beams or the column must be provided with a footing, to distribute the pressure.

Tests on compression across the grain are often made with the pressure distributed over only part of one of the loaded surfaces. Thus for tests on small specimens at the Forest Products Laboratory, a 2×2×6-in. block is used and loads are applied over the lower 2×6-in. surface and the middle third of the upper surface. Although such procedure does not give the true cross-grained compressive strength, it more nearly approaches the loading condition ordinarily met in a structure. In such tests, as the load is increased, the upper bearing closes the cell cavities immediately beneath it and gradually indents the surface. Beam action in the upper fibers often produces splitting in planes perpendicular to the line of pressure, but there is no well-marked failure.

Strictly speaking, timber does not have a well-defined elastic limit, since it takes set after the imposition of low loads. Nevertheless, the initial portion of the stress-deformation curve is approximately straight and it has become customary to record the stress corresponding to the limit of proportionality as the elastic limit. This is the only value of importance in tests across the grain.

Values of the compressive strengths of 2×2-in. prisms of 49 woods in green condition appear in Table 1. In Table 2, similar values may be found for larger sizes of the common structural timbers when air-seasoned. Table 3 shows the relation of air-seasoned to green material. Among the species having greatest compressive strength parallel to the grain we note osage orange, hickory, tanbark oak, longleaf pine, white ash, and redwood. Rock elm, maple, several varieties of oak, bald cypress, and shortleaf pine form a second class having good strengths.

For most of the conifers the compressive elastic limit across the grain varies between 10 and 15 per cent of the values determined for pressures parallel to the grain. Among the hardwoods the elastic limit in compression across the grain bears a higher ratio to the strength in compression parallel to the grain, the ratio being about 25 per cent for several

\* Record, *Mechanical Properties of Wood*, p. 15.

great that much difficulty is encountered in designing end connections so that the tensile strength of a piece can be developed. Therefore, wood tension members are rarely used. On this account and because the tensile strength parallel to the grain is so much greater than the compressive strength that the latter governs the strength of beams, the tensile strength with the grain is rarely tested.

The tensile strength parallel to the grain is influenced to some extent by the nature of the wood elements and their arrangement, but principally by the straightness of the grain and the thickness of the walls of the longitudinal elements. When failure occurs these elements are ruptured transversely. Knots greatly reduce the tensile strength parallel to the grain and are a great menace to strength when present in timbers subjected to such stresses (see Art. 240). The tensile strength appears to be less affected by moisture than are other mechanical properties.

In the following table the ratio of the tensile to the compressive strength has been tabulated for small specimens of several woods. The moisture content does not appear in the original publication, *Bull.* 10, U. S. Dept. of Agric., but it is probable that the pieces were air-dry.

Species.	Ratio. Tensile strength, Compr. strength	ULTIMATE STRENGTH IN Lb. PER SQ. IN.	
		Tension.	Compression.
Hickory.....	3.7	32,000	8500
Elm.....	3.8	29,000	7500
Larch.....	2.3	19,400	8600
Longleaf pine.....	2.2	17,300	7400

Across the grain, the tensile strength of wood is small. It is a property closely related to cleavability, and it often determines the strength of a beam which has cross-grain or spiral-grain in the tension fibers. Failure in tension across the grain occurs through separation of the cells and fibers in longitudinal planes. Knots, shakes, and checks all reduce the tensile strength of wood across the grain. The form of specimen used by the Forest Products Laboratory in making the test is shown in Fig. 1.

An examination of Table 1 shows that the tensile strength of wood across a radial plane is less than the tensile strength across a plane tangent to the rings. This difference is especially pronounced in the oaks and other hardwoods having large medullary rays. It is probable, therefore, that these rays considerably weaken the tensile strength of wood across a radial plane.

For the conifers, from which most structural timbers are secured, the cross-grained tensile strength of small perfect specimens of green wood runs between 200 and 325 lb. per square inch, while for the oaks it varies

between 600 and 1000 lb. per square inch. The smallness of these values must be remembered in computing the safe strengths of beams having cross-grain in the tension fibers.

### 229. The Shearing Strength of Wood.

—Although shearing stresses are often of small moment in metal beams and other structural elements, they are frequently of very great importance in members made of wood. Thus the horizontal shear stress at the neutral axis of a short, deep wooden beam may be sufficiently great to produce a failure like that of Fig. 2. If the mortise-and-tenon joint, shown in Fig. 3, is loaded so that the tenon presses downward upon the mortise, transverse shear stresses are produced on the tenon; and shear stresses parallel to the grain are caused in the vertical piece on planes *AC* and *BD*. In most cases failure in the tenon is due to bending rather than shear, but the pushing out of the piece *ABCD* is an example of shear failure.

Since the transverse shearing strength of wood is more than half the compressive strength parallel to the grain, it is very rarely necessary to design against failure through this stress. Tenons, handles of axes, hammers and mauls, and wooden pins are perhaps the most common examples of wooden pieces which must be proportioned against transverse shear. Not much experimental data on the transverse shear strength of timber has been published, and the results in Table 4 are of interest mainly in showing the variation in the strengths of pieces from trees of different species. The individual values should not be considered representative of the different species.

If the shearing stresses act on planes tangent to the growth rings, the resistance of the various woods is quite small, ranging from about 600 to 1000 lb. per square inch for small green specimens of the structural conifers and from 1000 to 1500 lb. per square inch for similar pieces of oak, hickory, elm, maple, sycamore, beech, birch, and white ash (see Table 1).

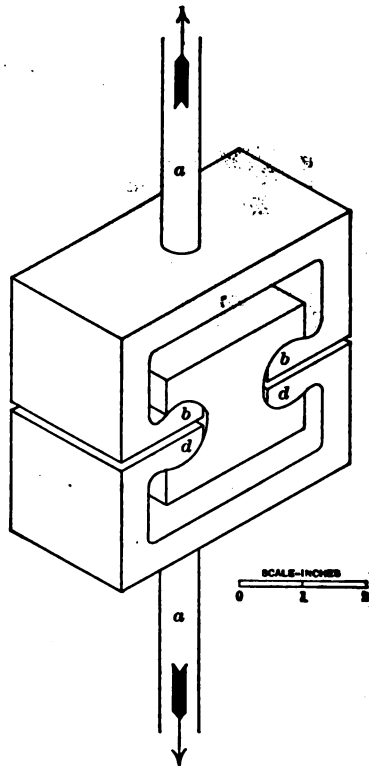


FIG. 1.—Type of Test Used by Forest Products Laboratory to Find Tensile Strength of Wood Across Grain. (Betts and Greeley before Int. Engr. Congress, 1915.)

In the same table it will be observed that there is little difference between the strengths of small pieces of green coniferous wood subjected to shear-



(Courtesy Forest Products Laboratory.)

Fig. 2.—Failure of a Large Wooden Stringer by Horizontal Shear.

ing parallel to the rings (tangential shear), and the strengths of like pieces subjected to shearing stresses acting on vertical radial planes (radial shear). Among the hardwoods the resistance to radial shear appears to be slightly the greater. In Fig. 4 are shown failures of pieces of green longleaf pine in radial shear (22) and in tangential shear (30).

TABLE 4.—THE RESISTANCE OF VARIOUS WOODS TO SHEAR ACROSS THE GRAIN

(J. C. Trautwine, in *Jour. Frank. Inst.*, Vol. 109, p. 106)

Test pieces were cylindrical pins  $\frac{1}{2}$  in. in diameter fairly seasoned and free from defects. They were subjected to double shear. Each result represents two tests.

Kind of Wood.	Shearing Strength (Lb.-In. <sup>2</sup> )	Kind of Wood.	Shearing Strength (Lb.-In. <sup>2</sup> )
Ash.....	6280	Locust.....	7176
Beech.....	5223	Maple.....	6355
Birch.....	5595	Oak, white.....	4425
Cedar, white.....	1446	Oak, live.....	8480
Cedar, Central American...	3410	Pine, white.....	2480
Cherry.....	2945	Pine, northern yellow.....	4340
Chestnut.....	1535	Pine, southern yellow.....	5735
Dogwood.....	6510	Pine, very resinous yellow.	5053
Ebony.....	7750	Poplar.....	4418
Gum.....	5890	Spruce.....	3255
Hemlock.....	2750	Walnut, black.....	4728
Hickory.....	6665	Walnut, common.....	2830

Turning our attention to the results in Table 5 we find the calculated horizontal shear stresses developed in tests of beams of structural sizes.

These shear stresses were computed from the formula  $S_h = \frac{3}{2} \frac{V}{a}$ ; where

$S_h$  = horizontal shear stress,  $V$  = maximum vertical shear and  $a$  = area of rectangular cross-section of beam. Comparing these results with the values for like woods found in Table 1, we observe that the computed stresses for the large beams are much lower than the shearing strengths of small specimens. This discrepancy is due principally to defects, shakes, and checks which cut down the area under shear. Since the formula assumes a full cross-section, the results given by it will be smaller than the stresses in the net sections of wood. The lesson taught is to use values from the tests on large beams failing in shear as a basis for design. Both Table 3 and 5 show that air-

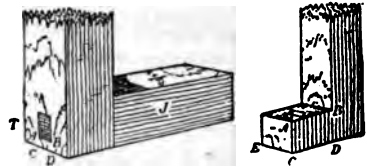


FIG. 3.—Wood Members Subjected to Longitudinal Shear.



FIG. 4.—Shear Failures Parallel to Grain in Green Longleaf Pine. Nos. 32 and 22, plane of shear radial, Nos. 30 and 34, plane of shear tangential to annual rings. (*Bulletin No. 70, Forest Service.*)

seasoned coniferous timber is, in most cases, stronger than green timber. Longleaf pine and redwood are, however, exceptions to this rule.

**230. The Strength of Wood in Cross-bending.**—Because of the great use of wood for beams, stringers, joists, rafters and other parts which are subject to bending, the cross-bending test is of much value in determining the quality of wood. By it one can measure the strength, toughness, and stiffness of the timber. Furthermore, the cross-bending test exposes weaknesses caused by defects better than does any other test. Unless very large testing machines are at hand, it is the only test which can be used to find the strength of timbers of structural sizes.

It will be remembered from Mechanics of Materials that the modulus of rupture in cross-bending is a fictitious measure of the ultimate unit



TABLE 5.—CALCULATED SHEARING STRESSES DEVELOPED IN STRUCTURAL BEAMS

(From Bull. 108, Forest Service)

Species.	TOTAL NUMBER OF TESTS.		FIRST FAILURE BY SHEAR.				SHEAR FOLLOWING OTHER FAILURE.				NO SHEAR FAILURE.			
	Green.	Dry.	Green.		Dry.		Green.		Dry.		Green.		Dry.	
			Per cent of total.	Average calculated shear per square inch.	Per cent of total.	Average calculated shear per square inch.	Per cent of total.	Average calculated shear per square inch.	Per cent of total.	Average calculated shear per square inch.	Per cent of total.	Average calculated shear per square inch.	Per cent of total.	Average calculated shear per square inch.
				Lb.		Lb.		Lb.		Lb.		Lb.		Lb.
Longleaf pine..	17	9	54	353	56	272	23	374	0	....	23	218	44	286
Douglas fir....	191	91	2	166	6	221	22	295	49	294	76	264	45	247
Shortleaf pine..	48	13	17	332	46	364	6	327	8	418	77	276	46	340
Western larch..	62	52	8	288	27	340	16	314	21	370	76	268	52	287
Loblolly pine..	111	25	7	335	28	434	2	356	16	546	91	237	56	255
Tamarack.....	30	9	10	261	33	299	3	263	0	....	87	228	67	283
Western hem- lock.....	39	44	5	288	23	307	28	281	68	438	67	250	9	417
Redwood.....	28	12	7	302	0	.....	11	218	17	250	82	283	83	227
Norway pine...	49	10	6	232	10	278	6	266	0	...	88	209	90	304

stress on the extreme fiber at the danger section. The modulus of rupture is computed from the formula  $S_m = \frac{M_m c}{I}$ ; where the modulus,  $S_m$  = unit stress on extreme fiber,  $M_m$  = maximum bending moment,  $c$  = distance from neutral axis to extreme fiber, and  $I$  = moment of inertia of the cross-section about the neutral axis. It will be recalled that the modulus of rupture as determined by this formula is intermediate between the tensile and compressive strengths and that it does not, therefore, truly represent the unit stress on the extreme fiber. The discrepancy between the calculated unit stress and the actual unit stress is due to the non-linear relation of stress to deformation when the elastic limit is exceeded, and also to the shifting of the neutral axis. The movement of the latter is brought about by the difference in the stress-deformation relations in tension and compression. Although a fictitious stress, the modulus of rupture is, nevertheless, a valuable index of quality of the wood. Also, if the elastic limit of the beam is determined, the ratio of the modulus of rupture to the unit stress on the extreme fiber at the elastic limit serves as a measure of the capacity of the beam to resist a momentary overload. Beams in which there is a well-marked difference between these stresses are preferable, since such members give ample warning of approaching failure.

The initial failure of long beams of uniform wood is indicated by a wrinkling of the overstressed compression fibers, much like the failure which occurs in compression prisms. Final failure of such beams is generally in tension. It is accompanied by more or less snapping as the individual fibers begin to break, and a load report when the maximum load is reached. Very dry specimens sometimes fail very suddenly in tension before any wrinkling of the compression fibers is noticeable. On the other hand, green test-pieces fail silently in compression without rupturing of the tensile fibers. Frequently, short deep beams fail by horizontal shear. This type of failure is sudden and is more common in well-seasoned timbers of structural sizes than in green timbers or in small beams. Very often shear failures result from defects.

Long, narrow beams must be restrained laterally and supported in such manner that the wide sides are vertical. If the first condition is not fulfilled, the member will fail from column action in the compression fibers. Unless the second condition is met the strength of the beam may be very seriously reduced through the inclination of the neutral axis. In laying joists for flooring these requirements are satisfied by diagonal bracing which the carpenter calls "bridging."

The relative strengths of the woods from various species of trees can be obtained from the values of the fiber stress at elastic limit and modulus of rupture which are tabulated in Table 1. It will be noted that the hardwoods as a class considerably exceed the conifers in static bending strength. Osage orange, honey locust, the various species of hickory, tanbark oak and white oak, rock elm, birch, beech and white ash are the stronger varieties of the hardwoods. Among the soft woods, longleaf pine, shortleaf pine, redwood, tamarack, and bald cypress are the strongest.\* Longleaf pine averages about the same in static bending strength as maple and red oak of the hardwoods. The ratios of the modulus of rupture to fiber stress at elastic limit range from 2.5 to 1.7 for the hardwoods and from 1.8 to 1.4 for the conifers. These ratios will be lower in beams of structural sizes because of defects.

In Table 6 strength values for cross-bending tests on air-seasoned timbers of structural size may be found. These results show very conclusively that the transverse strengths of small timbers are proportion-

\* Bamboo is a very strong wood in cross-bending, as the results of 8 tests by J. B. Johnson show. He used pieces which varied in diameter (between joints) from 0.54 in. to 1.25 in., and in thickness from 0.08 in. to 0.17 in. The spans varied from about 28 in. for the larger pieces to 8 in. for the smaller. Values of the modulus of elasticity ranged from 1,960,000 to 3,020,000 and averaged 2,380,000 lb. per square inch. The moduli of rupture varied from 19,600 to 41,100 and averaged 27,400 lb. per square inch. The average elastic limit was 17,300 lb. per square inch. All computations were based on the properties of the annular section. The average elastic resilience in inch-pounds per pound of weight of specimen was 216.

ately much higher than the strengths of large timbers. These discrepancies, which for the tests cited amount to 50 per cent in a number of instances, are largely due to the greater uniformity of the small test-pieces and their freedom from defects. Selected structural timbers (Grade I) in green state are, in general, about three-fourths as strong as small, clear pieces taken from them.

TABLE 6.—AVERAGE STRENGTH VALUES FOR AIR-SEASONED TIMBER BEAMS

(From Forest Service *Bull.* No. 108)

Species.	Sizes.		Number of tests.	Moisture.	Rings per inch.	Fiber stress at elastic Limit.		Modulus of rupture.		Modulus of elasticity.		Calculated shear.	
	Cross-section.	Span.				Average per square inch.	Ratio to 2 by 2 inches.	Average per square inch.	Ratio to 2 by 2 inches.	Average per square inch.	Ratio to 2 by 2 inches.	Average per square inch.	Ratio to 2 by 2 inches.
	In.	In.		P.ct.		Lb.		Lb.		1000 Lb.		Lb.	
Longleaf pine...	8×16	180	5	22.2	16.0	3390	.50	4,274	0.37	1747	1.00	288	0.75
	6×16	132	1	23.4	17.1	3470	.51	6,610	.57	1501	.86	388	1.01
	6×10	177	2	19.0	8.8	4560	.68	7,880	.68	1722	.99	214	.56
	4×11	180	1	18.4	23.9	3078	.46	8,000	.69	1660	.95	251	.66
	6×8	177	6	20.0	13.7	4227	.63	8,196	.71	1634	.94	177	.46
	2×2	30	17	15.9	13.9	6750	1.00	11,520	1.00	1740	1.00	383	1.00
Douglas fir.....	8×16	180	91	20.8	13.1	4563	.68	6,372	.61	1549	.91	269	.64
	5×8	180	30	14.9	12.2	5065	.76	6,777	.65	1853	1.09	218	.52
	2×2	24	211	19.0	16.4	6686	1.00	10,378	1.00	1695	1.00	419	1.00
Shortleaf pine...	8×16	180	3	17.0	12.3	4220	.54	6,030	.50	1517	.85	398	.98
	8×14	180	3	16.0	12.3	4253	.55	5,347	.44	1757	.98	307	.76
	8×12	180	7	16.0	12.4	5051	.65	7,331	.60	1803	1.01	361	.89
	5×8	180	6	12.2	22.5	7123	.92	9,373	.77	1985	1.11	301	.74
	2×2	30	67	14.2	13.7	7780	1.00	12,120	1.00	1792	1.00	404	1.00
Western larch...	8×16	180	23	18.3	21.9	3343	.57	5,440	.53	1409	.90	349	.96
	8×12	180	29	17.8	23.4	3631	.62	6,186	.60	1549	.99	295	.81
	5×8	180	10	13.6	27.6	4730	.80	7,258	.71	1620	1.04	221	.61
	2×2	30	240	16.1	26.8	5880	1.00	10,254	1.00	1564	1.00	364	1.00
Loblolly pine....	8×16	180	14	20.5	7.4	4195	.81	6,734	.72	1619	1.10	462	1.45
	6×16	126	4	20.2	5.0	2432	.47	4,295	.46	1324	.90	266	.84
	6×10	174	3	21.3	4.7	3100	.60	6,167	.66	1449	.99	173	.54
	4×12	174	4	19.8	4.7	2713	.52	5,745	.61	1249	.85	185	.58
	8×8	180	9	22.9	4.9	2903	.56	4,557	.48	1136	.77	93	.29
	6×7	144	2	21.1	5.0	2990	.58	4,968	.53	1286	.88	116	.36
	4×8	132	8	19.5	9.1	3384	.65	6,194	.66	1200	.82	196	.62
	2×2	30	123	17.6	6.6	5170	1.00	9,400	1.00	1467	1.00	318	1.00
Tamarack.....	6×12	162	5	23.0	15.1	3434	.45	5,640	.43	1330	.82	318	.75
	4×10	162	4	14.4	9.7	4100	.54	5,320	.41	1356	.84	252	.59
	2×2	30	47	11.3	16.2	7630	1.00	13,080	1.00	1620	1.00	425	1.00
Western hemlock	8×16	180	44	17.7	17.8	4398	.69	6,420	.62	1737	1.04	406	1.06
	2×2	28	311	17.9	19.4	6333	1.00	10,369	1.00	1666	1.00	382	1.00
Redwood.....	8×16	180	6	26.3	22.4	3797	.79	4,428	.57	1107	.96	294	1.05
	6×12	180	6	16.1	17.7	3175	.66	3,353	.43	728	.64	167	.60
	7×9	180	6	15.9	15.2	3280	.69	4,002	.51	1104	.96	147	.53
	3×14	180	6	13.1	24.4	.....	.....	5,033	.64	.....	.....	291	1.04
	2×12	180	5	13.8	14.4	3928	.82	5,336	.68	1249	1.09	260	.93
	2×10	180	5	13.8	24.8	3757	.79	4,606	.59	1198	1.05	186	.67
	2×8	180	6	13.7	20.7	4314	.90	5,050	.65	1313	1.15	166	.60
	2×2	28	122	15.2	18.8	4777	1.00	7,798	1.00	1146	1.00	279	1.00
Norway pine...	6×12	162	5	16.7	8.1	2968	.56	5,204	.61	1123	.97	286	1.02
	4×10	162	5	13.7	12.0	5170	.98	6,904	.82	1712	1.48	317	1.13
	2×2	30	60	14.9	11.2	5280	1.00	8,470	1.00	1158	1.00	281	1.00

Tiemann \* variations of 25 per cent in the above rates will not affect strength more than 2 per cent.

In order that proper factors of safety may be established on a basis of testing machine results, it is very desirable that the strength of timber under dead loads be determined. A few tests on small beams by the late Dr. R. H. Thurston, indicated that 60 per cent of the progressively applied ultimate load would break beams if left in place for nine months. In Fig. 5 appear the results of approximately seventy-five end compression tests by J. B. Johnson on  $1\frac{1}{8} \times 1\frac{1}{8} \times 3$ -in. prisms. All specimens were cut from a single plank of longleaf pine which was ripped into prisms 40 in. long. The specimens were cut from each long prism as indicated in Fig. 5 and dressed to the above size. All tests were made in a 30,000-lb. universal machine. For dead-load tests the prisms were bedded on a nest of four vertical car springs which deformed about an inch under the imposed loads. By means of this elastic base the yielding of the specimen was taken up and load of almost constant intensity maintained throughout the test. Each plotted point in Fig. 5 represents three to six tests. The horizontal asymptote to this curve would probably be somewhere between the 50 and 60 per cent ordinates. Consequently, as a result of the above tests, it is not safe to assume that the permanent load which timber will carry is greater than 50 per cent of the short-time ultimate as ordinarily found by the testing machine.

### STIFFNESS AND OTHER MECHANICAL PROPERTIES

**232. The Stiffness of Wood.**—Stiffness in a structure is often as of much importance as strength, but it is much more frequently neglected in designing. Floors must be sufficiently stiff so that they will not deflect appreciably under working loads or else they give one the feeling of insecurity. If a floor sustains a plastered ceiling its deflection under working load should not exceed  $\frac{1}{360}$  of the span. Likewise the deflection of rafters should be limited, if it is desirable to avoid the disagreeable appearance of a sagged roof.

Stiffness in compression members is not often of moment in design. It is measured by the modulus of elasticity which is computed from the ratio of unit stress to unit strain. For beams the modulus of elasticity ( $E$ ) may be computed from the equation  $E = \frac{Pl^3}{\beta f I}$ ; where  $P$  = a certain load within the elastic limit of the beam,  $f$  = deflection corresponding to  $P$ ,  $l$  = length of span,  $I$  = moment of inertia of cross-section about the neutral axis and  $\beta$  = a constant depending on the end conditions and the

\* For a more complete discussion see *Proc. A. S. T. M.*, Vol. 8, p. 541.

method of loading the beam. For a center load on a simple beam,  $\beta=48$ ; for the case of a simple beam loaded with two equal concentrated loads ( $P, P$ ) at the third points of the span,  $\beta=\frac{648}{23}$ .

In general, the denser woods are the stiffer, as may be seen by reference to Table 1. There is not, however, much difference between stiffness of the softwoods as a class and the stiffness of the hardwoods, nor are there as wide variations in the stiffnesses of the various species of wood as in the strengths. Values in Table 3 show that green timber is less stiff than air-seasoned; and, from results in Table 6, it appears that the structural sizes of timber are about as stiff as the small clear sticks.

Fig. 6 shows typical load-deflection and load-deformation curves for wood. These figures also indicate the method of finding the elastic limit and the correction of curve when it does not pass through the origin. Other load-deformation curves for wood may be seen in Fig. 18.

**233. Toughness.**—A wood which has a large capacity to resist shocks or blows is called tough. The spokes of an automobile or wagon, the tongue of a wagon and its axles, the handle of the axe or sledge all must be tough that they may absorb without injury the shocks which they receive. In order to be tough a wood must have both strength and flexibility.

Toughness is best measured by the energy of the blow required to rupture a beam in transverse impact. A less reliable index of toughness is obtainable from the energy of rupture in cross-bending. The latter test, however, is more easily made than the former and of more general value, since static strength and stiffness may also be gotten from it. The torsion test has also been used to measure the toughness of wagon spokes.

In Table 1 the average work done in deforming a large number of small wooden specimens in both static and impact bending tests has been recorded. The methods of determining the various results in static bending have been considered in Ch. I and III. In the impact tests the height of drop was increased by 1- or 2-in. intervals until failure took place. The height of drop at the elastic limit was gotten by plotting height of drop ( $h$ ) against the square of the deflection ( $f$ ) and determining the value  $h'$  at which the curve deviated from a straight line. The fiber stress

at elastic limit ( $S_e$ ) was calculated from  $S_e=\frac{3Wh'l}{fbd^2}$ ; the modulus of elasticity ( $E$ ) from  $E=\frac{l^2 \times S_e}{6hf}$ ; and the elastic resilience ( $K$ ) from  $K=\frac{Wh'}{lbd}$ . The undefined symbols are  $W$ =weight of hammer,  $l$ =length of span,  $b$ =breadth of beam, and  $d$ =depth of beam.

From the test results on green timber, it will be observed that the total

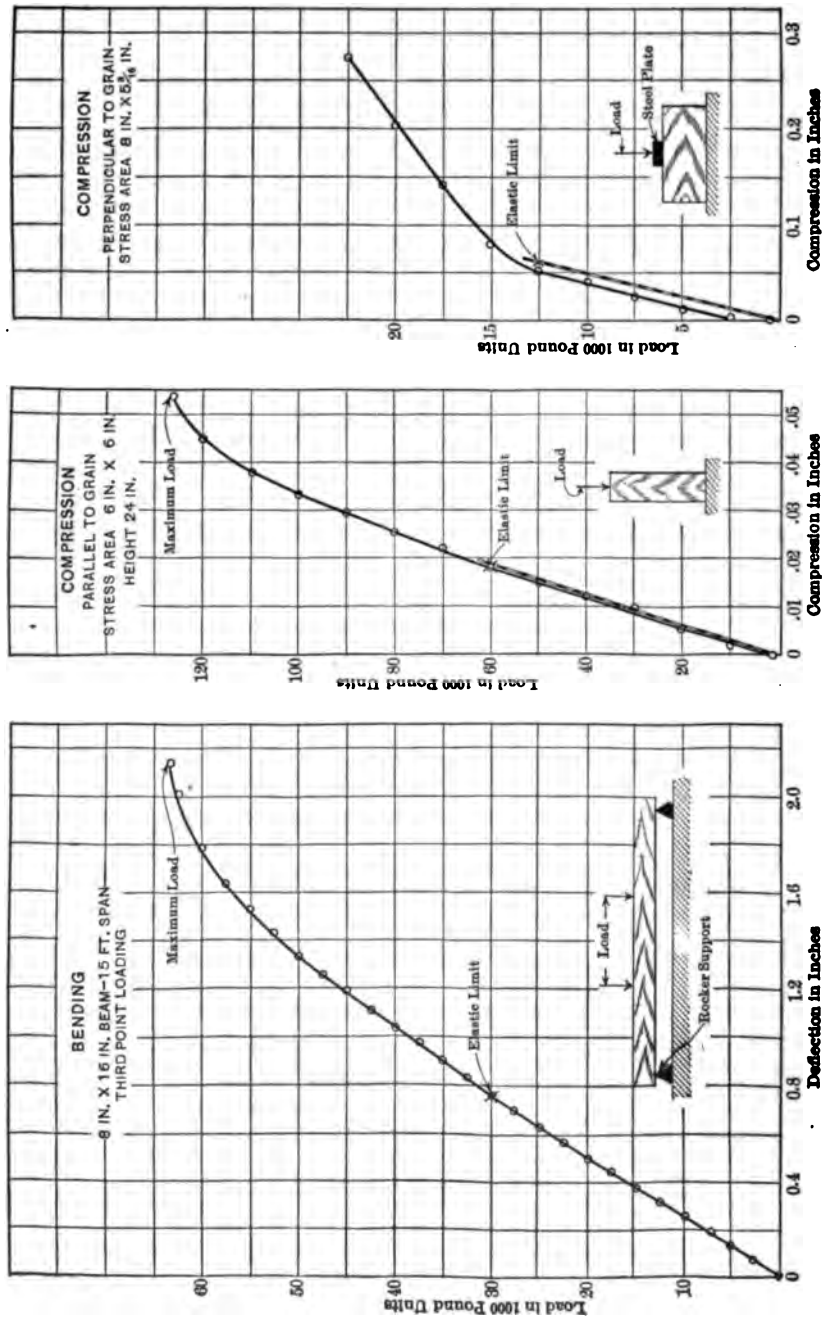


Fig. 6.—Representative Stress Deformation Diagrams for Bending, Compression Parallel to the Grain, and Compression Perpendicular to the Grain. (From *Bulletin No. 108*, Forest Service.)

work in static bending and the height of drop causing complete failure in impact bending vary with different species in approximately the same manner. With these calculated values as criteria, it is obvious that the hardwoods as a class excel the conifers in toughness. Among the hardwoods, osage orange, hickory, rock elm, slippery elm, honey locust and hackberry are very tough; while basswood and sycamore are more brittle than many of the softwoods. Longleaf pine is the only one of the conifers possessing much toughness.

Seasoning when unaccompanied by checking generally increases toughness, but in chestnut, gum and willow it causes a marked decrease in toughness and, to a less extent, adversely affects hickory. In general, greenwood is tougher than seasoned material.

**234. Cleavability** is the measure of the ease with which wood may be split. This property is of considerable moment in the working of wood, especially in splitting fence rails and firewood. Woods which must be fastened by nails and screws should have a high resistance to splitting.

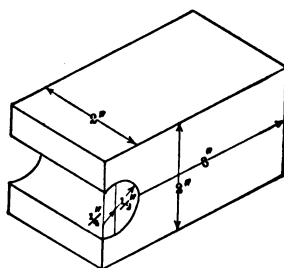
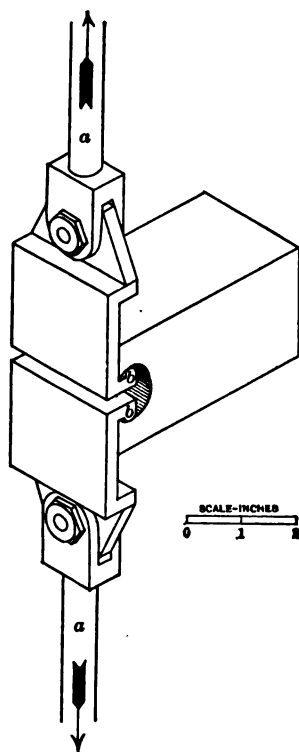


FIG. 7.—Type of Cleavability Test and Specimen Adopted by Forest Products Laboratory. (Betts and Greeley before Int. Congr. Congress, 1915.)

Since splitting is accomplished by wedging apart the longitudinal elements, it is closely related to tension, across the grain. At the Forest Products Laboratory, the test-piece of Fig. 7 is used to determine resistance to cleavage.

From results in Table 1, it appears that most hardwoods split more easily along radial planes than along tangential surfaces. Among the conifers the difference in cleavage strength in the two directions is not great, but for longleaf pine, hemlock and tamarack it is greatest across radial planes. Interlocking of the wood fibers causes high cleavage strength,

while defects like shakes and checks reduce it. Knots may affect it either way depending upon the number, position and character.

Honey locust, hickory, slippery elm, hard maple, and the oaks have the highest resistance to splitting. Basswood and the conifers split with comparative ease.

**235. Hardness.**—Both resistance to indentation and resistance to scratching are important properties in woods which are to be used for finishing and for furniture. These properties together with the ability to wear without splintering determine the wearing resistance of wood for floors and pavements. Aside from the indentation tests no satisfactory type of test has been devised to measure these properties. However, experience shows that woods having marked difference in the character of the spring wood and summer wood (pine and oak), wear best when laid with the edge of the grain exposed to wear. With the fine-grained woods of uniform texture, like hard maple, the resistance to wear appears to be little affected by the method of sawing.

The resistance to indentation of a number of green woods is given in Table 1. With the exception of basswood, all of the hardwoods listed are harder than long leafpine, the hardest of the conifers. In green wood the hardness appears to be independent of the surface indented. Seasoning greatly increases resistance of all surfaces to indentation but affects the resistance of the end surfaces most.

#### CONDITIONS AFFECTING MECHANICAL PROPERTIES OF TIMBER

**236. Density.**—All of the mechanical properties of clear wood, regardless of species, are related to its density. Within a given species the relation is closer than between species. This is most pronounced in timbers of structural size where defects and moisture content considerably affect interspecies relationships. Thus, from a very large number of tests at the Forest Products Laboratory, it appears that for small specimens of green material, the shrinkage, the compressive strength parallel to the grain, and the stiffness vary with the first power of the density; the shearing strength parallel to the grain and static bending strength vary as slightly higher power of density; whereas, cleavage, work in bending, tension across the grain, hardness, and compression perpendicular to the grain vary approximately with the second power of the density.

The general law may be expressed by  $Q = C\rho^n$ ; where  $Q$  is the property considered,  $C$  is a positive constant,  $n$  is a positive constant between 1 and  $2\frac{1}{2}$ , and  $\rho$  is the density. In such relationships  $\rho$  is computed from the volume of the wood at the time of testing and the oven dry weight.

Fig. 8 shows the relation between modulus of rupture and the dry weight per cubic foot for 113 varieties of wood. In Fig. 9 the relation between





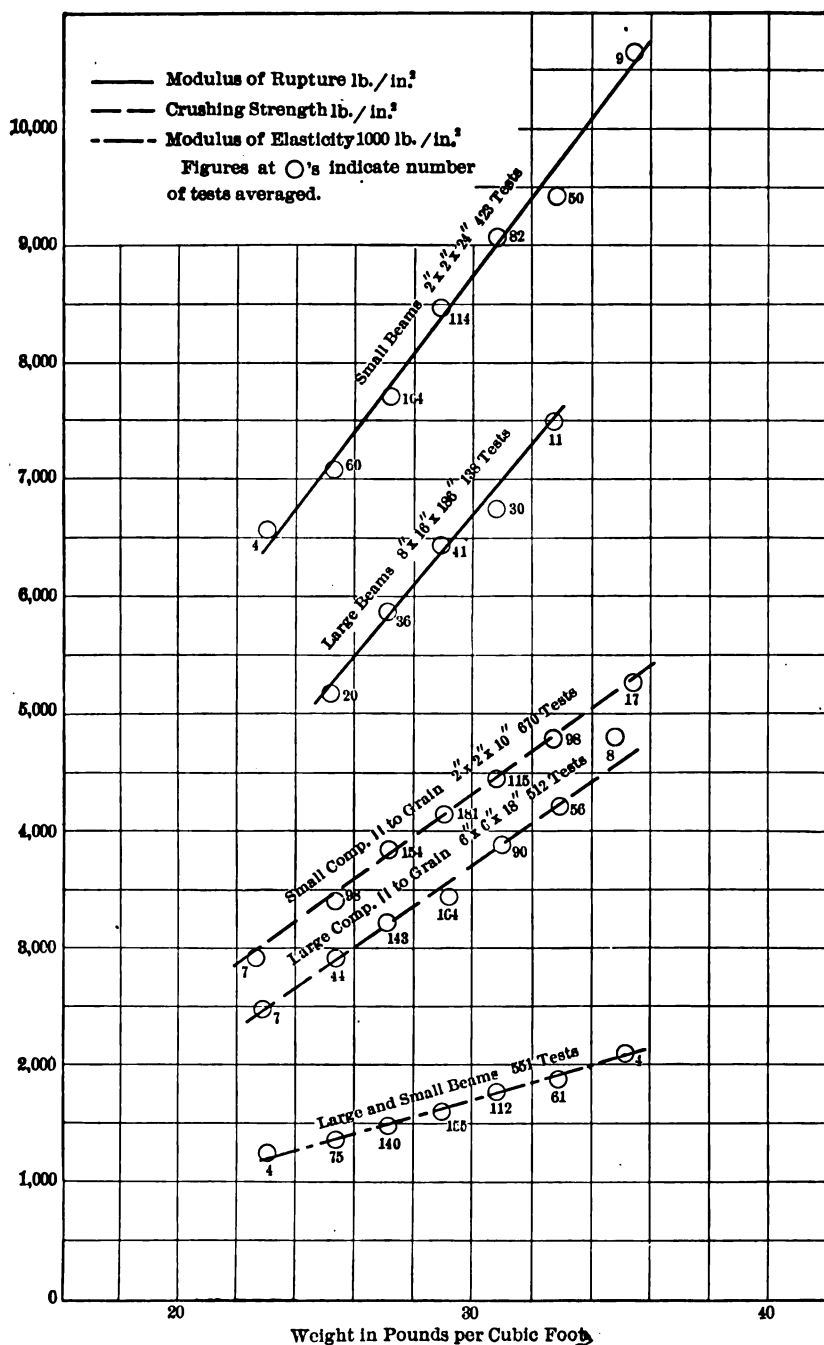


FIG. 9.—Relation of Dry Weight to Modulus of Rupture and Modulus of Elasticity in Bending, and to Compressive Strength Parallel to Grain. (Bull. No. 88, Forest Service.)

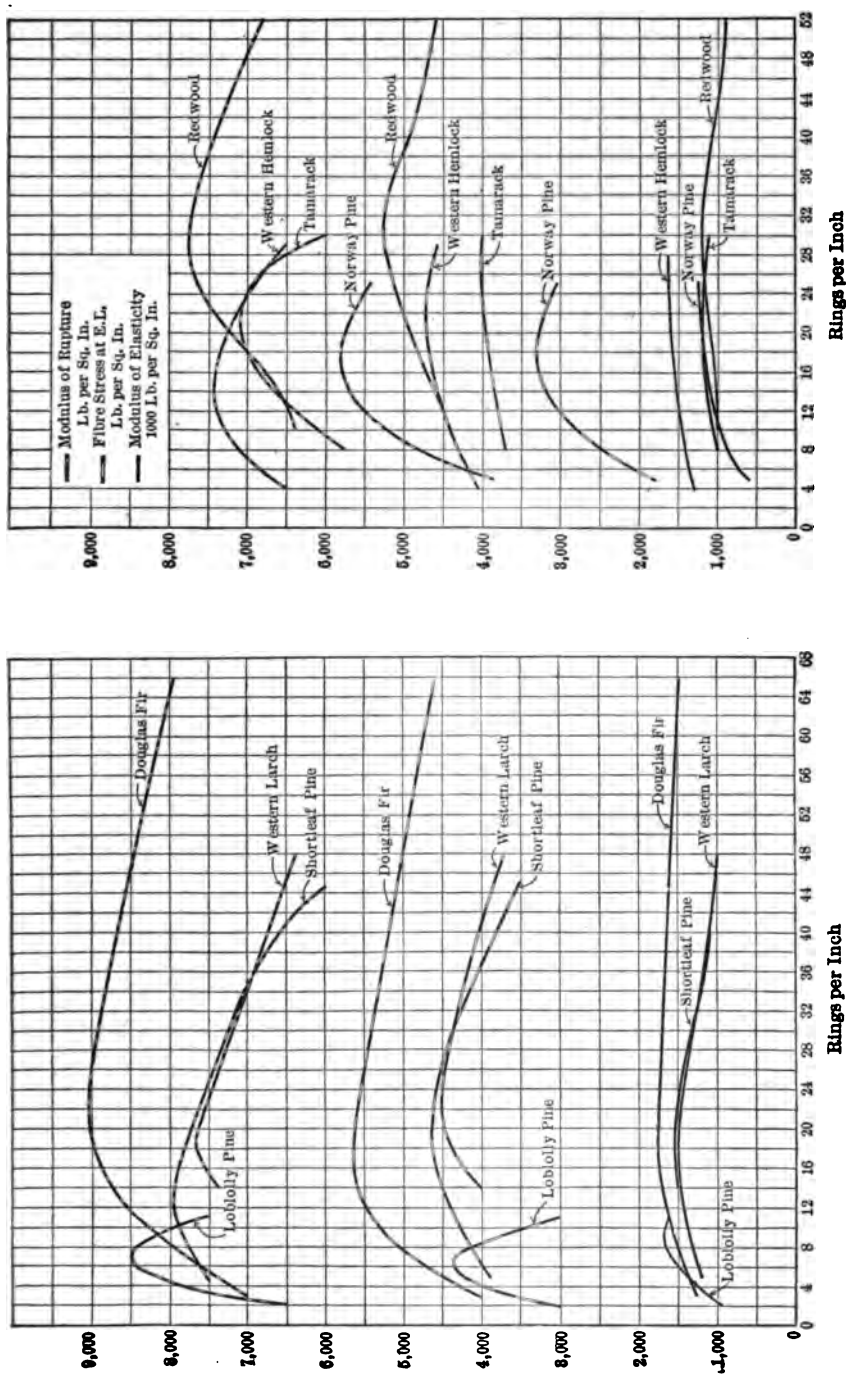


Fig. 10.—Relation of Modulus of Rupture, Fiber Stress at Elastic Limit, and Modulus of Elasticity to Rate of Growth, as Shown by Number of Rings per Inch. (From Bull. 108, Forest Service.)

various mechanical properties of air-seasoned beams of Douglas fir and the dry weight is shown. Attention should be directed to the difference

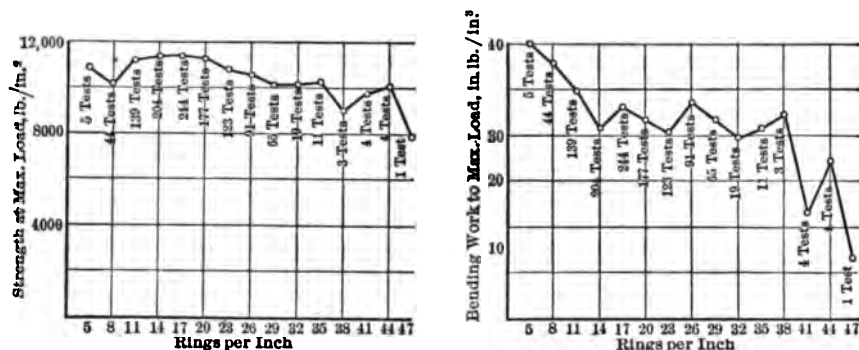


FIG. 11.—Bending Strength and Energy of Rupture as Affected by Rate of Growth in Green Hickory. (*Bulletin No. 80, Forest Service.*)

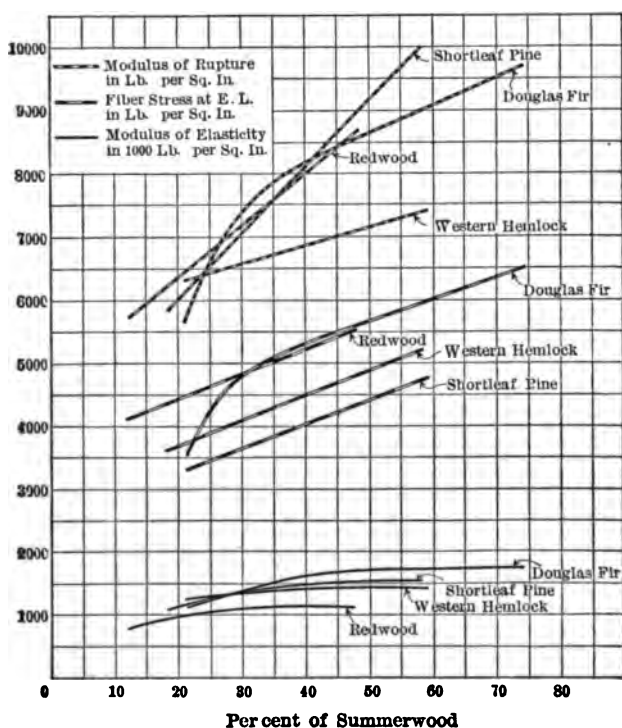


FIG. 12.—Relation of Modulus of Rupture, Fiber Stress at Elastic Limit, and Modulus of Elasticity to Percentage of Summer Wood. (From *Bull. No. 108, Forest Service.*)

in strength values for the large and small specimens. A considerable proportion of these discrepancies is due to defects which were present in

the large beams. It is also noteworthy that the stiffness is independent of the size of specimen. From tests on other species the latter conclusion appears to be general.

Since density is affected by rate of growth, per cent summer wood, position of specimen in tree, and moisture content, the influence of these factors on mechanical properties will next be considered.

**237. Effect of Rate of Growth.**—Fig. 10 illustrates the previously made statement that coniferous wood having a medium rate of growth is the strongest and stiffest. There is, however, a wide range in the most effective rates of growth for different conifers. In the dense hardwoods rapid growth is more desirable as is shown by the curves of Fig. 11.

**238. Effect of Percentage of Summerwood.**—With most coniferous wood the summer wood is readily identified and forms a valuable index of the mechanical properties of the timber. The proportion of summer wood in a given coniferous timber is generally determined by estimating the ratio of the sum of the areas of the dark rings to the total cross-section.

Fig. 12 shows the relation of per cent summer wood to strength and stiffness for several coniferous woods. On account of the important relation which the per cent summer wood bears to mechanical properties a minimum percentage limit has been inserted in specifications for structural timber (see Art. 250).

**239. Relations of Mechanical Properties to Position in Tree.**—Since wood in the lower part of the trunk of a tree is more dense than that higher up, and since the densest wood at any given height is situated between the pith and the middle ring of the cross-section, a small variation in the strength of wood due to position in the tree will be found. Fig. 13 shows how the strength and energy of rupture of hickory specimens vary due to position in tree. Fig. 14 shows how the strength of wood in Western larch trees varies with the height above ground.

**240. The Influence of Defects on Mechanical Properties.**—Defects

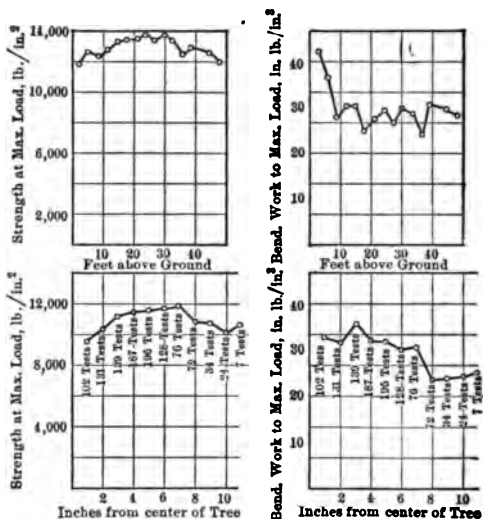


FIG. 13.—Bending Strength and Work as Affected by Position in the Tree, as Shown by Tests on Green Hickory. (From *Bull. No. 80, Forest Service.*)

are one of the principal causes of variation in the mechanical properties of timber. The discrepancies in the strength values of large and small specimens and the variation in properties of similar test-pieces from the same wood are conditions largely due to defects.

Knots destroy the continuity of the wood elements and consequently

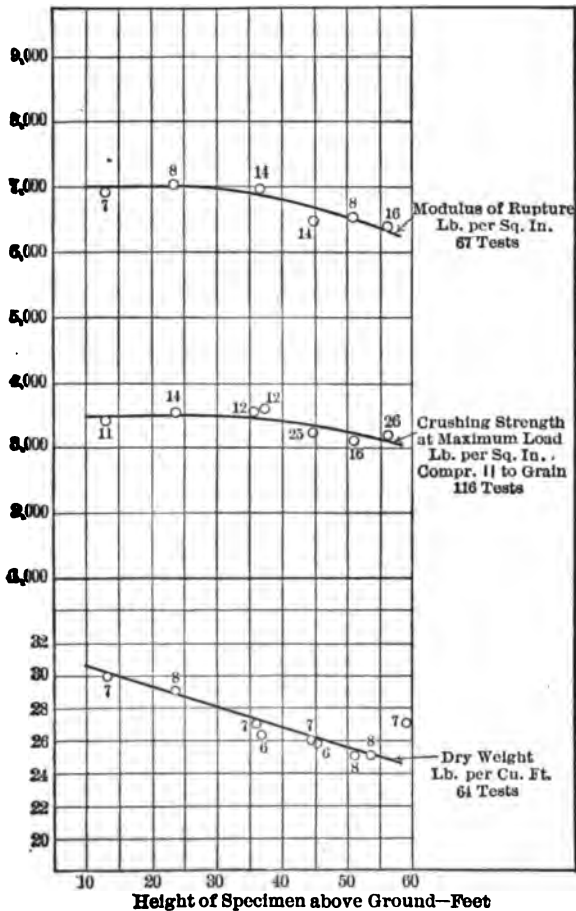


FIG. 14.—Relation of Strength Values to Height in Tree. Specimens were small clear pieces of green western larch. (*Bull. No. 122, Forest Service.*)

diminish tensile strength. Such defects are, therefore, a source of weakness when present in the lower fibers of a simple beam, and especially so if under the load. Large knots and knots which are inclined to the axis of a member adversely affect the strength of a column or, if in the central portion of the top fibers, weaken a beam. Table 7 shows the effects of knots on the compressive strength and stiffness of 6×6-in. prisms of



Douglas fir, Western hemlock, and Western larch. From the results in this table it appears that knots  $1\frac{1}{2}$  in. in diameter or over may diminish the compressive strength and stiffness of structural timbers from 15 to 20

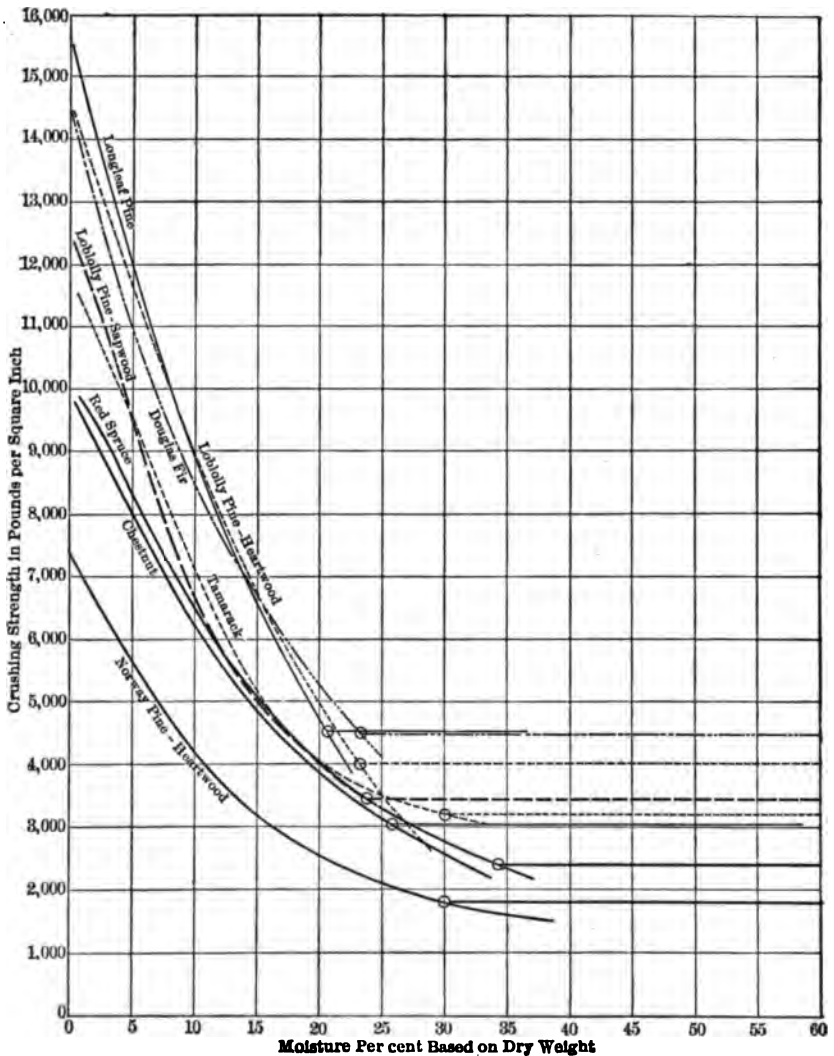


FIG. 15.—Relation between the Crushing Strength Parallel to Grain and the Moisture Content for Several Woods. (Circular No. 108, Forest Service.)

per cent. Tests by the Forest Products Laboratory have shown that knots have little effect on the elastic limits and stiffnesses of beams but they decrease the modulus of rupture. Consequently knots in beams will



adversely affect ultimate strength and toughness. Sound knots near the neutral plane have little influence on the shearing strengths of beams.

Shakes and checks are most harmful to strength when they follow the neutral plane of a beam or run diagonally across the tension side of it. In the first case they weaken the resistance to horizontal shear, and in the second case they lessen the tensile strength.

Restrictions on the character, size, and position of defects in structural timbers are given in the grading rules proposed by the Forest Products Laboratory (see Art. 250).

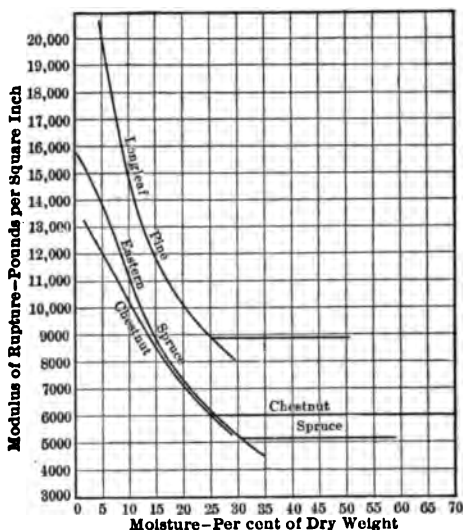


FIG. 16.—Variation of Strength in Bending with Moisture Content of Wood. (From Bull. No. 70, Forest Service.)

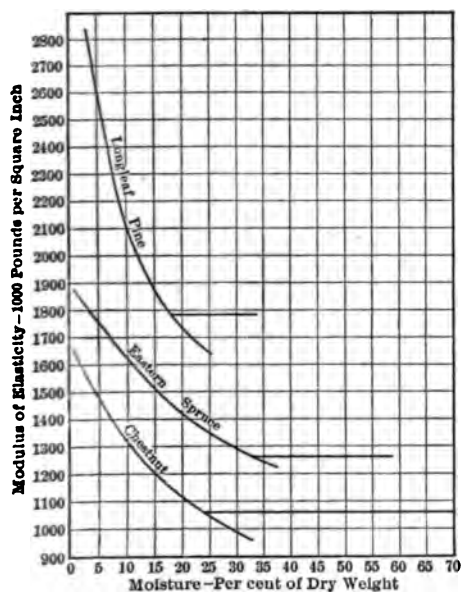


FIG. 17.—Relation between Stiffness in Bending and Moisture Content for Three Species of Wood. (From Cir. No. 108, Forest Service.)

**241. The Effect of Moisture on Mechanical Properties.**—Variations in the moisture content of the cell walls are accompanied by large changes in the strength and stiffness of wood. These effects are most noticeable in small, clear pieces which season with great rapidity. On account of the slowness with which large structural timbers dry it is, however, unsafe to count on any increase in strength due to air-drying of such members. After years of seasoning, large timbers may lose enough water to effect an increase in tensile and compressive strength and in stiffness, but defects arising from shrinkage stresses often cause a decrease in the resistance to horizontal shear stresses. In kiln-drying the normal increase

in strength due to loss of moisture is often nullified by a condition which prevents complete drying of the piece and produces stresses. Since, however, pieces less than  $4 \times 4$  in. in cross-section are considerably strengthened and stiffened by proper drying, the effect of the influence of moisture on these properties will now be

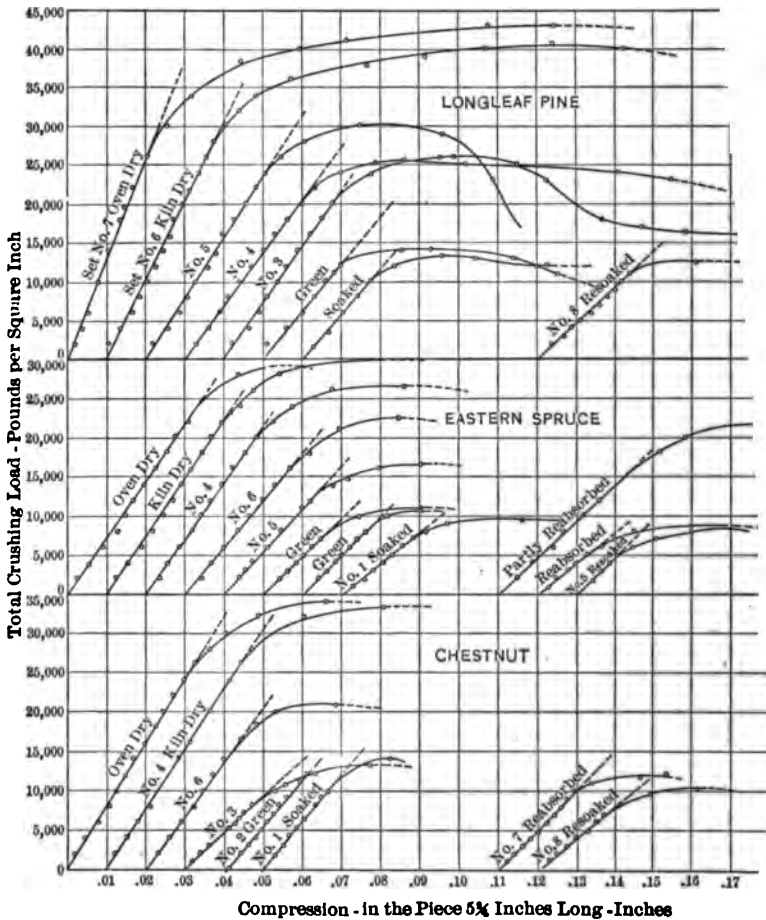


FIG. 18.—Stress-deformation Diagrams for Compressive Tests Parallel to the Grain in Woods Varying in Moisture Content from Wet to Oven Dry Condition (Bull. No. 70. Forest Service.)

The mechanical properties of wood are not materially affected by a reduction in the moisture content until the fiber-saturation point is reached. Further drying causes a large proportionate increase in strength and stiffness. These statements are confirmed by evidence in Figs. 15, 16, and 17. Fig. 18 shows the effects of variations in moisture content on

very pliable and is weakened. The results of a large number of tests on the effect of various drying and steaming treatments on wood were reported by Tiemann in the *Lumber World Review*, Apr. 10, 1915.

From this report, Table 9 has been abstracted. The results show that steaming green wood under high temperatures and pressures greatly weakens it. Air-dried wood subjected to temperatures above 330° F. and steam pressures of 90 lb. per square inch was also weakened but to a less extent than the soaked wood.

TABLE 9.—THE WEAKENING EFFECT OF VARIOUS DRYING PROCESSES ON THE CRUSHING STRENGTH OF 2×2-IN. WOOD PRISMS

(Tiemann, in *Lumber World Review*, Apr. 10, 1915)

Treatment.	Kind of Wood.	* REDUCTION IN STRENGTH IN PER CENT OF NORMAL WHEN		
		Soaked, Treated, Soaked, Tested.	Air-dried, Treated, Air-dried 1 Year, Tested.	Soaked, Treated, Air-dried 1 Year, Tested.
Dry air, 145° F., 25 days.....	White ash	-23.4	0	0
	Loblolly pine	-21.0	0	
	Red oak	- 7.9	+16.3	
Saturated steam 212° F., 1 hr.....	White ash	-12.5	0	0
	Loblolly pine	-24.8	0	
	Red oak	+13.8	- 4.4	+ 1.1
Saturated steam 212° F., 4 hr.....	White ash	-10.3	0	0
	Loblolly pine	-19.7	0	
	Red oak	- 1.5	- 2.4	
Saturated steam 90 lb./in. <sup>2</sup> , 331° F., 5 min....	White ash	-24.8	4	0
	Loblolly pine	-27.3	- 9	
	Red oak	-44.5	-49.1	-13.7
Saturated steam 90 lb./in. <sup>2</sup> , 331° F., 1 hr.	White ash	-44.7	-27	0
	Loblolly pine	-36.6	-42	
	Red oak	-39.9	-61.3	-24.9
Superheated steam 0 lb./in. <sup>2</sup> , 331° F., 6 hr. ....	White ash	-11.9	0	-3
	Loblolly pine	-18.0	0	
	Red oak	-49.8	-60.0	
10 lb./in. <sup>2</sup> , 274° F., 4 hr.....	White ash	-11.9	0	-3
	Loblolly pine	-18.0	0	
	Red oak	-49.8	-60.0	

\* The minus sign indicates a loss in strength.

Prof. W. K. Hatt \* states that a high degree of steaming reduces the strength and the spike-holding power of the wood. The safe limiting conditions for steaming depend upon the quality of the wood, the thoroughness of seasoning, the steam pressure, and the duration of steaming. He states that loblolly pine should not be steamed at pressures above 30 lb. per square inch for more than four hours, or 20 lb. per square inch for more than six hours.

\* Circular No. 39, Forest Service.

**243. The Effect of Preservatives on Strength.**—Experiments made at the Forest Products Laboratory \* show that creosote oil in itself is not injurious to the strength of timber, but excessive heat or pressure during the steaming and impregnation processes may injure strength considerably. From a boiling treatment (see Art. 220), on Douglas fir timbers, both green and air-seasoned stringers lost about one-third of their normal strength. Similar results were gotten on green Douglas fir stringers which were subjected to a full-cell process with a preliminary steaming at 325° F. In a number of tests on small specimens of Douglas fir, longleaf pine, and shortleaf pine, the results showed that air-dry material was less affected by treatment than green wood. Several methods of creosoting were used, but the temperature was kept below 200° F. and the pressure below 150 lb. per sq. in. The bending strength, stiffness, crushing strength, and longitudinal shear resistance of specimens of air-dry material were not materially injured by treatment; whereas similar green specimens were weakened, in some cases over 30 per cent. In tension across the grain both air-dry and green specimens lost heavily in strength by treatment; in some cases over half of the normal strength was thus destroyed.

Zinc chloride solutions of the strengths commonly used in treating do not seriously weaken timber, but apparently do render it somewhat more brittle under impact.

**244. Fire-killed Douglas Fir.**—From the results of over 400 tests on beams of structural size and several hundred tests on smaller specimens,† it appears that fire-killed Douglas fir, if sound, is practically as strong and just as stiff as material gotten from growing trees. If such timber has stood for several years after the fire, it is likely to be badly eaten by beetles, borers, and fungi. Consequently careful inspection should be made to detect evidences of such deterioration before using fire-killed Douglas fir.

**245. Effect of Bleeding on Strength of Longleaf Pine.**—The wood of pine trees which have been bled for turpentine is just as strong as that of unbled trees. This statement is based upon the results of over 1300 tests made by the Forest Service (*Cir.* No. 12) on air-dry material. Furthermore, these tests show that neither the shrinkage nor the specific weight are affected by bleeding; and that the resin loss, confined to the sapwood, is insufficient to affect the durability of the wood. Other tests indicate that the strength of longleaf pine is independent of the resin content.

\* See *Circular* No. 39, of Forest Service, and *Bulletin* No. 286 of U. S. Dept. of Agric.

† J. B. Knapp in *Forest Service Bull.* No. 112.

## STRENGTH OF NAILS IN WOOD

**246. Holding Force of Nails.**—One of the most valuable properties of wood is the facility with which boards may be attached by means of nails and the strength of such attachments. The holding force of nails and spikes in different woods is, therefore, of considerable importance. In Fig. 19, the starting resistances against the drawing out from dry oak wood of nails having different styles of points, are shown graphically.

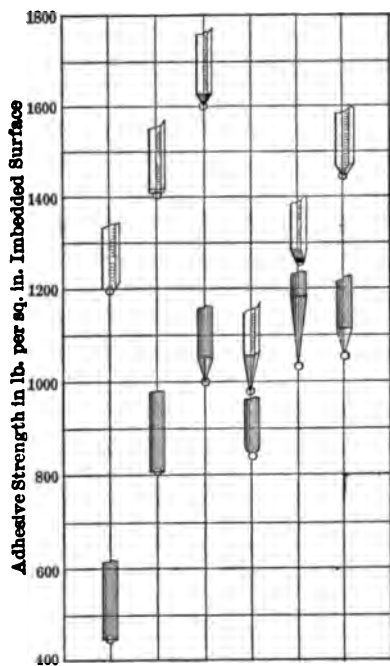


FIG. 19.—Relative Adhesive Strength of Wire and Cut Nails in Oak Wood as Affected by the Shapes of their Points. (*Engr. News*, Vol. 31, p. 24.)

The cut nails exhibit a much greater holding force than do the wire nails, and a slightly sharpened point gives the highest resistance for each species. This figure exhibits the holding force of different nails, per square inch of embedded surface, when driven laterally into dry oak wood. Evidently for the softer woods the resistance to drawing is very much less, and so is the resistance when driven endwise into the stick.

**247. Holding Force of Railroad Spikes.**—In Table 10 are given results of tests made at the University of Illinois to determine the holding force of ordinary spikes and screw spikes in various woods. The common spikes were 5½ and 6 in. long by ⅞ to ¾ in. square; they were fairly smooth. The screw spikes varied from 5 to 5½ in. in length and had a diameter at root of thread of ¾ to ⅝ in. The thread projected ⅛ to ⅜ in. and the pitch was ½ in. Spike holes were bored ⅞ in. in diameter, thereby insuring a tight fit. The results in the table are

corrected for a uniform depth of 5 in. Other tests made to determine the effect of depth of penetration on strength indicated that these factors were directly proportional. Tests to determine the effect of form of point on resistance to withdrawal showed that blunt-pointed and bevel-pointed spikes were slightly superior to chisel-pointed spikes. Common spikes when redriven had about 60 to 80 per cent of the resistance of newly driven spikes.

Not only is the holding force of screw spikes much greater than that

TABLE 10.—RELATIVE HOLDING POWER OF SCREW SPIKES AND ORDINARY SPIKES IN SEVERAL TIMBERS

(Bull. No. 6, Engr. Expt. Sta., University of Illinois)

Kind of Tie.	Kind of Spike.	RESISTANCE IN POUNDS FOR			RELATIVE RESISTANCES.		
		½-in. Pull.	¾-in. Pull.	Max. Resist.	½-in. Pull.	¾-in. Pull.	Max. Resist.
Water oak. . . . .	Ordinary..	2870	5,730	6,780	100	100	100
	Screw. . . .	4888	9,180	12,190	170	160	179
Black oak. . . . .	Ordinary..	2910	5,890	7,230	100	100	100
	Screw. . . .	4760	10,420	14,110	164	177	203
Red oak. . . . .	Ordinary..	2950	5,350	7,730	100	100	100
	Screw. . . .	4900	10,400	13,560	166	194	176
White oak. . . . .	Ordinary..	3510	5,950	7,870	100	100	100
	Screw. . . .	6250	11,900	12,630	178	200	188
Ash. . . . .	Ordinary..	3570	5,200	7,730	100	100	100
	Screw. . . .	5700	10,470	12,760	162	200	165
Beech. . . . .	Ordinary..	2600	5,490	8,840	100	100	100
	Screw. . . .	6450	13,140	16,230	248	221	238
Elm. . . . .	Ordinary..	2380	5,580	7,500	100	100	100
	Screw. . . .	5120	10,090	13,690	215	181	183
Poplar. . . . .	Ordinary..	2830	5,290	5,670	100	100	100
	Screw. . . .	3880	6,210	7,490	137	117	132
Chestnut. . . . .	Ordinary..	2850	4,070	5,200	100	100	100
	Screw. . . .	3690	6,340	8,700	129	155	167
Sweet Gum. . . . .	Ordinary..	3230	4,120	5,300	100	100	100
	Screw. . . .	5430	7,710	8,280	167	162	156
Loblolly pine. . . . .	Ordinary..	2920	3,500	4,300	100	100	100
	Screw. . . .	5750	9,050	10,620	197	258	247

of plain spikes, but their resistance to lateral thrust is also considerably higher. Prof. W. K. Hatt \* found that the lateral resistance of screw spikes was 1.2 to 2.4 times the resistance of plain spikes similarly tested. No support was provided for the head of the spike in these tests. In timber treated with crude oil the holding force of plain spikes is considerably less than in untreated timber or in timbers treated by other commer-

\* See Bull. No. 124, Am. Ry. Engr. Assn.

cial processes. For shortleaf pine treated with crude oil, Hatt found the holding force of plain spikes to be only 45 per cent. and of screw spikes 73 per cent. of the values obtained for the respective spikes driven in untreated wood.

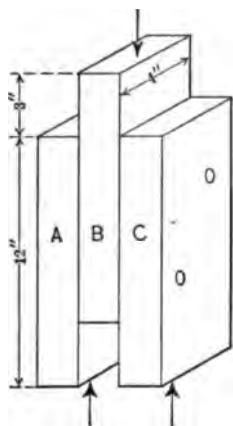


FIG. 20.—Type of Joint Used in Shear Tests on Common Wire Nails.

#### 248. The Shearing Strength of Nailed Joints.

—A considerable number of experiments at Iowa State College\* yielded the results given in Table 11. The joints were tested as shown in Fig. 20. The center-piece (B) in the joint was in all cases sufficiently thick so that the nails after piercing the outer planks would not pass through it. An equal number of nails was driven from each side of the joint and the ends of the blocks were made plane to provide a square bearing. Although the report states that true shear was obtained without tension or bending moment due to eccentricity, it is obvious that in any test of this character bending of the nails must take place especially after the yield point of the joint has been passed. The amount of bending and its influence on the strength will depend largely upon the bearing strength of the wood.

TABLE 11.—STRENGTH OF COMMON WIRE NAILS IN SINGLE SHEAR

(Bull. No. 2, Vol. 4, Iowa State College)

NAILS USED.		Length of Nails, In.	Gauge of Wire.	AVERAGE STRENGTH PER NAIL FOR TOTAL NUMBER OF NAILS TESTED, POUNDS.				
No. Penny.	No. of Nails Tested.			White Pine.	Yellow Pine.	Spruce.	Fir.	Oak.
8	20	2½	10½	230	294	187	478	656
10	20	3	9	235	324	237	475	608
12	8	3½	9	318	625	332	360	665
16	20	3½	8	299	494	263	456	750
20	20	4	6	372	746	386	632	992
30	20	4½	5	540	1221	629	832	1392
40	20	5	4	656	1067	555	837	1580
50	12	5½	3	712	1425	563	985	1925
60	12	6	2	826	1783	794	.....	2125

The following conclusions were deduced from these tests. The variation from the mean in the shearing strength for single nails was less than 38 per cent and averaged 6 per cent. In general it appeared from these tests that the shearing strength per nail was independent of the number of

\* Bull. No. 2, by M. I. Evinger, Engr. Expt. Sta., Iowa State College.

in the joint and varied directly with the size of the nail and the depth of penetration. Barbed nails in 8- and 10-penny sizes, gave 40 per cent greater strength to yellow pine joists than smooth wire nails of like size. In soft non-resinous woods lost from 6 to 16 per cent of their strength in seasoning, but hardwood joints showed an increased strength of 20 to 32 per cent when seasoned. Joints made from wood having a loose fiber texture were weaker than those fashioned from dense material.

### WORKING STRESSES AND GRADING RULES

**49. Working Stresses.**—It is probable that less reason and more common sense has been used in stipulating factors of safety for timber than in any other building material. Undoubtedly the lack of reliable data on mechanical properties, unsatisfactory methods of inspecting and grading, and the necessity of providing against decay have been the main considerations causing the use of high factors of safety for timber.\*

Wood has also been considered to be a very variable material, and so when compared to structural steel. Tests, however, show that it is as uniform as concrete, brick, or stone. For example, consider the results of experiments on 32 species of wood comprising over 5000 tests of small compression specimens and a like number of small beams which were reported in *Circular No. 15*, of the Forest Service. They show that percentage of crushing strengths within 25 per cent of the average ranged from 65 to 100 per cent for different species and averaged 93 per cent for all species. In cross-bending from 58 to 95 per cent of the specimens had strengths within 25 per cent of the species' averages, the average for all species being 76 per cent. Further results of bending tests by the Forest Service † in which nine species and 575 structural timbers in green condition were experimented upon, show that only 3 to 13 per cent of the timbers of any one species fell below 75 per cent of the average strength. If the grading rules eliminate material which is unsatisfactory through variation of density or through inherent defects, most of these weak timbers could be discarded by proper inspection. In view of these facts and in view of the data now accumulated it appears safe to use higher working stresses in bending and in end compression than have been advocated heretofore.

Table 12 gives suggested working unit stresses for structural timbers in bending. The values of working stresses in bending are based on a minimum factor of safety of approximately  $1\frac{2}{3}$  for long-time loads on the best timbers which would be passed by the grading rules, or a factor of about  $2\frac{1}{4}$  for average timber under the same kind of loading. Based on

\* See standard texts on Mechanics; Merriman's, Rankine's and others.

† *Bulletin No. 108*, Figs. 4 to 12.



testing machine determinations of moduli of rupture for small, clear sticks of green wood the average factor of safety lies between 5 and 6. For highway bridges these stress values should be diminished one-sixth and for railway bridges and trestles one-third. It should be borne in mind in using these stresses that they apply to timbers with defects limited (see Grading Rules), and that a dry location is presupposed. If conditions surrounding the structure favor decay, the timber should be treated with preservatives; or, only the more resistant species should be used and a minimum amount of sapwood together with a lower working stress should be specified.

TABLE 12.—ALLOWABLE WORKING UNIT STRESSES FOR STRUCTURAL TIMBERS IN BUILDING CONSTRUCTION \*

Stresses are in pounds per square inch

[Prepared from data furnished by the Forest Products Laboratory.]

Species.	BENDING.		COMPRESSION.	
	Stress in Extreme Fiber.	Horizontal Shear Stress.	Parallel to Grain on Short Columns.	Perpendicular to Grain
Oak.....	1400	125	900	400
Southern yellow pine (dense).....	1600	125	1200	350
Southern yellow pine (sound).....	1300	105	1000	300
Douglas fir (dense).....	1600	100	1200	350
Douglas fir (sound).....	1300	90	1000	300
Western hemlock.....	1300	75	900	300
Eastern hemlock.....	1000	70	700	300
Norway pine.....	1100	85	800	300
Eastern spruce.....	900	70	600	200
Eastern white pine.....	900	80	700	250
Tamarack.....	1200	95	900	350
Balsam fir.....	900	75	700	150

\* Unless otherwise specified the values apply only to timber which passes the grading rules of the Forest Products Laboratory for defects.

**250. Grading Rules.**—As an example of a means for grading structural timber in accordance with its mechanical properties a portion of the rules of the American Society for Testing Materials for Southern yellow pine have been inserted. On account of the difficulty in discriminating longleaf from other pines, the species classification has been abandoned and limitations on density and defects have been made the bases of classifying.

*Southern Yellow Pine:* This term includes the species of yellow pine growing in the Southern states from Virginia to Texas, that is, the pines hitherto known as longleaf pine (*Pinus palustris*), shortleaf pine (*Pinus echinata*), loblolly pine (*Pinus taeda*), Cuban pine (*Pinus heterophylla*), and pond pine (*Pinus serotina*).

Under this heading, two classes of timber are designated: (a) dense southern yellow pine and (b) sound southern yellow pine. It is understood that these two terms are descriptive of quality rather than of botanical species.

(a) Dense southern yellow pine shall show on either end an average of at least six annual rings per inch and at least one-third summer wood, or else the greater number of the rings shall show at least one-third summer wood, all as measured over the third, fourth, and fifth inches on a radial line from the pith. Wide-ringed material excluded by this rule will be acceptable, provided that the amount of summer wood as above measured shall be at least one-half.

The contrast in color between summer wood and spring wood shall be sharp and the summer wood shall be dark in color, except in pieces having considerably above the minimum requirement for summer wood.

(b) Sound southern yellow pine shall include pieces of southern pine without any ring or summer wood requirement. (See Sec. 3, p. 1, Standard Definitions of Terms Relating to Structural Timber, Serial Designation: D 9-15.)

This society specifies that standard dimension for bridge timbers shall be within  $\frac{1}{4}$  in. of the specified size when rough, and within  $\frac{1}{8}$  in. when dressed.

The following extract from the A. S. T. M. specifications for bridge timbers gives the grading rules for stringers. These rules are to be coupled with the above density requirement. The Specifications (Serial Designation: D 10-15) also include requirements for caps, sills, posts, longitudinal struts and girts, and longitudinal X-braces, sash braces and sway braces.

(a) *Dense Southern Yellow Pine:* Dense southern yellow pine shall show not less than 80 per cent of heart on each of the four sides, measured across the sides anywhere in the length of the piece; loose knots, or knots greater than  $1\frac{1}{2}$  in. in diameter, will not be permitted at points within 4 in. of the edges of the piece.

(b) *Sound Southern Yellow Pine:* Sound southern yellow pine shall be square-edged, except it may have 1 in. wane on one corner. Knots shall not exceed in their largest diameter one-fourth the width of the face of the stick in which they occur. Ring shakes extending not over one-eighth of the length of the piece are admissible.

The Forest Products Laboratory proposes a rule which is similar to the above excepting that it sharply limits the position of defects in beams. The parts of the proposed rule dealing with defects may be applied to all species of structural timber; for Douglas fir the clauses relating to contrast between spring wood and summer wood should be omitted.

#### RULES PROPOSED BY FOREST PRODUCTS LABORATORY FOR GRADING STRUCTURAL TIMBERS OF SOUTHERN YELLOW PINE \*

##### GRADE I

Requirements for quality, number of rings, and proportion of summer wood:

1. Shall contain only sound wood.
2. Wood shall have a density indicated by the following characteristics:

\* From a paper "Structural Timber in the United States," by H. S. Betts and W. B. Greeley, presented at Int. Engr. Congress at San Francisco, Cal., Oct. 1, 1915.

Shall show on the cross-section an average of not less than one-third summer wood, measured over the third, fourth, and fifth inches on a radial line from the pith. Timber with an average of less than six annual growth rings per inch shall show an average of not less than one-half summer wood. Contrast in color between summer wood and spring wood shall be sharp.

In cases where timbers do not contain the pith, and it is impossible to locate it with any degree of accuracy by curvature of the rings, the same inspection shall be made over three inches of an approximately radial line beginning at the edge nearest the pith.

**3. Restrictions on Knots in Beams.**—Shall not have in Volume I (see Fig. 21) sound knots with a diameter greater than  $1\frac{1}{2}$  in. or one-fourth the width of the face on which they appear, or knots over one-half inch in diameter which are insecurely attached to the surrounding wood. Shall not have in Volume II sound knots with

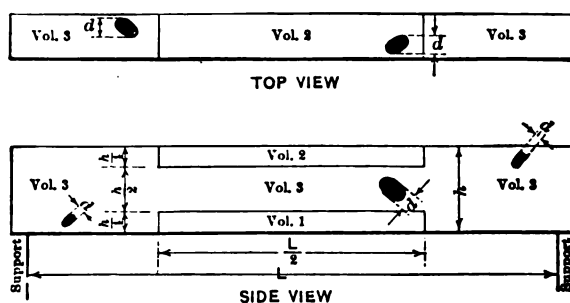


FIG. 21.

a diameter greater than 3 in. or one-half the width of the face on which they appear, or insecurely attached knots with a diameter greater than  $1\frac{1}{2}$  or one-fourth the width of the face on which they appear.

The aggregate diameter of all knots within the center half of the length of any face shall not exceed the width of that face.

**NOTE.**—The diameter of a knot on the narrow or horizontal face of a beam is to be taken as its projection on a line perpendicular to the edge of the timber. On the wide or vertical face the smallest dimension of a knot is to be taken as its diameter. See Fig. 21.

**4. Restrictions on Knots in Columns.**—Sound knots whose diameters are greater than 4 in., or one-third the least width of the column, or knots over  $\frac{1}{2}$  in. in diameter which are insecurely attached to the surrounding wood shall not be permitted.

**NOTE.**—The diameter of a knot is to be taken as its projection on a line perpendicular to the edge of the column. See Fig. 21.

**5. Restrictions on Shakes and Checks in Beams.**—Ring-shakes shall not occupy, at either end of a timber, more than one-fourth the width for green material, nor more than one-third the width for seasoned material.

Any combination of checks and shakes which would reduce the strength to a greater extent than the allowable ring-shakes will not be permitted. Shakes shall not show on the faces of either green or seasoned timber.

**NOTE.**—The importance of shakes and checks in the middle half of the height of a beam is dependent upon the magnitude of the horizontal shearing stress. The specification given is for beams in which the allowable shearing stress is developed. In beams whose length is more than 15 times their height the allowable shearing stress is not usually developed and as the ratio of length to height increases,

importance of these shakes and checks decreases proportionately. Ring-shakes, showing on an end, shall be considered as extending to the center. Checks and radial or star-shakes are not usually serious in the direction of the length of the timber but should not be allowed if, in the judgment of the inspector, their weakening effect is greater than that of the allowable ring-shakes. Allowable conditions for both green and seasoned material are given because checks are formed and are further developed during seasoning. The maximum amount of shakes and checks should be allowed unless it is practically certain from the appearance of the timber, or from a knowledge of seasoning conditions or length of time elapsed since cutting, that it is thoroughly seasoned. For uses of this rule it will be considered that at least  $1\frac{1}{2}$  months per inch of thickness is required for material to become thoroughly seasoned.

4. **Restrictions on Cross-Grain in Beams.**—Shall not have diagonal grain with a greater than 1 in 20 within the middle half of the length of the beam.

## GRADE II

Grade II includes material rejected from Grade I on account of either (A) having density than required for Grade I or (B) having more serious defects than allowed for Grade I.

(A) Material rejected from Grade I because of deficient density will be accepted for Grade II, provided it conforms to all the requirements of Grade I, except that in paragraph 2 the requirements for *one-third* summer wood in material having 6 rings over per inch, shall be changed to *one-fourth*; and that the requirements for *one-third* summer wood in material having less than 6 rings per inch, shall be changed to *one-fourth*.

(B) Material rejected from Grade I for having excess of serious defects will be accepted in Grade II, provided it conforms to paragraph 2 of Grade I, and has defects limited as follows:

7. **Restrictions on Knots in Beams.**—Shall not have in Volume I sound knots with a diameter greater than 3 in. or one-half the width of the face on which they appear; or knots which are insecurely attached to the surrounding wood, whose diameter exceeds  $1\frac{1}{2}$  in., or one-half the width of the face on which they appear. Shall not have in Volume II knots which are insecurely attached whose diameter exceeds 3 in., or one-half the width of the face on which they appear. The aggregate diameter of all knots within the center half of the length of any beam shall not exceed two times the width of that face.

*NOTE.*—For method of measuring diameters of knots in beams see note in rule 3.

8. **Restrictions on Knots in Columns.**—Sound knots whose diameters are greater than 6 inches or one-half the least width of the column, or knots which are insecurely attached to the surrounding wood and whose diameters are greater than 3 in. or one-fourth the least width of the column shall not be permitted.

*NOTE.*—For method of measuring diameters of knots in columns see note in rule 4.

9. **Restrictions on Shakes and Checks in Beams.**—Ring-shakes shall not occupy either end of a timber more than one-third the width of the beam, nor more than one-half the width for seasoned material.

Any combination of checks and shakes which exceeds the allowable ring-shakes shall not be permitted. (See also rule 5.)

Grading rules for the hardwood lumber industry, by the U. S. Commerce Department, 1911.

## CHAPTER VII

### BUILDING STONE \*

**251. Uses and Production.**—We shall include under the head of building stone those stones which are used for ornamentation or for any form of masonry construction, as in foundations, retaining walls, buildings, bridges, pavements. In some of these constructions stone has been employed as a building material since the first Egyptian dynasty. The tombs and pyramids of Egypt, the excavated ruins of ancient Troy, remains of Grecian and Roman structures, and the many old monuments of stone scattered through England, France, Germany, and the Holy Land, all bear witness to the use which the ancients made of this material. The conditions which govern in the selection of stone for structural purposes are cost, fashion, ornamental value, and durability; although the latter property, it must be admitted, is frequently overlooked or disregarded. Cost is largely influenced by transportation charges, difficulties in quarrying and cutting, the ornamental features, and the durability of the stone. Color and arrangement, and shape of mineral constituents greatly influence fashion and ornamental value. Resistance to fire and power to withstand weather conditions—factors which are largely influenced by the mineral constitution of the rock—are the most important determinators of durability.

Suitable stone for structural purposes is widely distributed both in this country and abroad. Granite is found in large quantities in Canada, England, Scotland, Sweden and Norway; within the United States Vermont, Massachusetts, Maine, California, Wisconsin and North Carolina, lead in production. Limestone and marble are found abroad in Canada, Italy, England, Algeria, Germany, Belgium, France and Austria; the following states lead in producing limestone: Pennsylvania, Indiana, Ohio, New York, Illinois and Missouri, while Vermont, Georgia, Tennessee, Alabama and New York provide most of the domestic marble. Sandstone is obtainable in Canada and Scotland; New York, Ohio, Pennsylvania, Washington and Massachusetts lead the states in production. Trap rock is produced principally in California, New Jersey, Pennsylvania, New York, Washington and Massachusetts. Slate is found abroad in

\* References freely consulted in preparing this chapter are Merril's *Stone for Building and Decoration*, Ries' *Building Stones and Clay Products*, Eckel's *Building Stones and Clays*, Ries' and Watson's *Engineering Geology*.

Canada, England and France: within the United States the principal sources are Pennsylvania, Vermont, Virginia, New York and Maryland.

In 1914, the value of building stone (exclusive of slate), produced in the United States was \$77,412,292. Of this amount the limestone production totaled approximately 44 per cent, the granite 26 per cent, marble 10 per cent, trap rock 10 per cent, and sandstone 10 per cent. The value of the stone sold for various purposes was as follows: 39 per cent, principally limestone and trap rock, was marketed as crushed stone; 23 per cent was sold as rough or dressed building stone; 9.1 per cent, granite and marble, was sold for monuments; 4.9 per cent, chiefly granite and sandstone, was the value of the material sold for paving; 4.8 per cent was sold for rip-rap; 2.4 per cent, largely sandstone and granite, was sold for curbing. Nearly eight million dollars' worth of limestone included in the above amount was sold as flux. The production of slate in 1915 for the United States was valued at \$4,958,915.

**252. The Mineral Constituents of Rocks.**—Inasmuch as rocks are aggregations of minerals, their properties will be dependent upon the character of these constituents. Valuable aid in determining minerals is afforded by the blowpipe and by the reactions of the minerals with acids. For identifying minerals, for ascertaining their condition, and for observing the structural arrangement in a rock, the examination of thin sections under the microscope by an expert petrographer is often of great value. Chemical analyses of rocks are of little importance except in corroborating microscopic observations. The rapid determination of mineral constitution is ordinarily made, however, through recognition of certain physical properties. The more important of these properties are hardness, cleavage, streak, color, luster, specific gravity and shape of crystals.

Hardness is probably of most importance for rapid determinations of minerals. It is measured by scratching the mineral with a series of substances of known variation in hardness, the following scale of Mohs being generally used:

1. Talc, easily scratches with the thumb-nail.
2. Gypsum, scratched by the thumb-nail.
3. Calcite, not scratched by thumb-nail, but easily cut by knife.
4. Fluorite can be cut by knife with greater difficulty than calcite.
5. Apatite can be cut only with difficulty by knife.
6. Orthoclase can be cut by knife with great difficulty on thin edges.
7. Quartz, not scratched by steel, scratches glass.
8. Topaz.
9. Sapphire.
10. Diamond.

If, for example, a given substance is scratched by calcite and not by gypsum its hardness is between 2 and 3.

Cleavage is the measure of the ease and distinctness with which a mineral may be separated on planes parallel to the crystal faces. All minerals do not exhibit cleavage and only a few possess well-marked cleavage. Easy, distinct, indistinct, perfect, and imperfect, are words often used to characterize cleavage.

Streak is the color of the mineral when in form of a powder. It is generally determined by rubbing the mineral on a bit of unglazed porcelain, called a streak stone. Since the streak of a mineral is constant in color, it is of considerable importance in distinguishing the hematites and the sulphides of metallic minerals, which vary considerably in color of mass.

Color is a valuable characteristic of the metals but is less a reliable index of the non-metallic minerals, especially if they are contaminated by coloring agents.

Luster is the surface appearance of a mineral under reflected light. It is frequently described by the following terms: Vitreous (glassy); greasy, pearly, resinous, dull, silky and metallic.

Crystal form is of importance when a mineral has had opportunity to develop its natural shape. This is not the normal condition in rock structure. Nevertheless the form is often of much importance in microscopic examinations.

In the following list will be found the most common mineral constituents of building stones together with their chemical constitution and more important physical properties.\*

*Quartz*, silicon dioxide, is a most valuable constituent of many rocks. Hardness = 7; sp. gr. = 2.66. No cleavage. It is a colorless, white to gray sometimes brown to black mineral with a white streak and vitreous luster. It is soluble in hydrofluoric acid only and weathers well. *Flint* is a non-crystalline form of silica.

*Feldspars* are a group of silicates of alumina with potash (*orthoclase*) or with soda or lime (*plagioclase*). Hardness = 6; sp. gr. 2.62 to 2.75. Cleavage is good on two mutually perpendicular planes. The streak is white and the luster vitreous to pearly. *Orthoclase* varies in color from deep pink to whitish pink, *plagioclase* from gray to white. *Feldspars* are less durable than quartz.

*Micas* are silicates of alumina with hydrogen and potash (*muscovite*) or with iron, magnesia and hydrogen (*biotite*). Hardness = 2 to 3; sp. gr. = 2.7 to 3.2. *Micas* may be split along one plane into very thin tough plates. Their luster is vitreous to pearly and their streak is colorless or gray. In color, *muscovite* is colorless or gray to brown; and *biotite* is brown to black. *Micas*, especially *biotite*, do not weather well.

*Amphibole* commonly occurs as *hornblende*, a complex silicate of iron, lime, magnesia and alumina, or sometimes as *tremolite*, a silicate of lime and magnesia. Hardness = 5 to 6; sp. gr. = 2.9 to 3.5. Cleavage is perfect on two planes 124° apart but the mineral does not separate into flakes like mica. Luster is vitreous and streak uncolored, grayish or brownish. *Hornblende* is dark green to black, *tremolite* is white to gray. *Hornblende* weathers fairly well, *tremolite* poorly.

*Pyroxene* is chiefly found as *augite*, a silicate of lime, alumina, magnesia and iron.

\* For further information see *Engineering Geology* by Ries and Watson or *Optical Mineralogy* by N. H. and A. N. Winchell.

Hardness=5 to 6; sp. gr.=3.2 to 3.6. Cleavage is good on two planes 93° apart. Color is green to black. Resembles hornblende and weathers fairly well.

*Olivine*, a silicate of iron and magnesia, is a round-grained, greenish mineral having a hardness of 6 to 7. Its cleavage is indistinct, luster vitreous, and streak uncolored. It weathers poorly.

*Chlorites* are a group of aluminum silicates containing iron and magnesia. They are of greenish color and have cleavage properties like mica, but less elastic leaves. Hardness=2 to 2.5. Streak is white to green.

*Garnets* are commonly silicates of iron and alumina; less often the silicates may contain lime or magnesia. Hardness=6.5 to 7.5. Cleavage is poor, streak white, color red, and luster vitreous. They render stone difficult to dress and polish.

*Serpentine* is a hydrous silicate of magnesia having a greenish color and soapy feel. Hardness is usually about 4. The streak is white and luster greasy. It is soluble in hydrochloric acid and has poor resistance to weather.

*Talc*, another hydrous silicate of magnesia, has a hardness of 1. Splits into thin, brittle plates. Color is white to green, luster pearly. *Soapstone* is a massive form of talc.

*Calcite*, calcium carbonate, effervesces in dilute cold hydrochloric acid. Its color is quite variable, white when pure. Hardness=3. Sp. gr.=2.72. Cleavage is perfect in three directions. Slowly dissolved by waters containing traces of acids; not very durable.

*Dolomite*, calcium-magnesium carbonate, resembles calcite. Hardness=3.5 to 4. Sp. gr.=2.85. Its color is often pink and white. Effervesces in hot dilute acid but not in cold. Less soluble than calcite but not very durable.

*Gypsum*, hydrous calcium sulphate, is a colorless or white mineral. Its hardness is 2; sp. gr. 2.32, cleavage perfect in one plane, streak white. It is soluble in hydrochloric acid and slightly so in water.

*Limonite*, a hydrous sesquioxide of iron, has no cleavage. Its streak is yellowish brown. Hardness=5 to 5.5. Sp. gr.=3.6 to 4. Color varies from yellow to dark brown. Soluble in hydrochloric acid.

*Magnetite*, a combination of ferrous and ferric oxides of iron, is a black, strongly magnetic mineral. Hardness=5.5 to 6.5; sp. gr.=5.16 to 5.18. Cleavage is indistinct, luster metallic, and streak black. Slowly soluble in hydrochloric acid. Rusts on weathering.

*Pyrite*, iron disulphide, is a very common impurity in rocks. Color is brassy yellow, streak green to black, luster metallic. It has no cleavage. Hardness =6 to 6.5; sp. gr.=4.95 to 5.1. Oxidizes readily when exposed to the weather.

## IMPORTANT STONES FOR STRUCTURAL PURPOSES

**253. Classes of Rocks.**—In accordance with geological origin, rocks may be classified as *igneous*, formed by the more or less rapid cooling of molten material from inside of the earth; *sedimentary*, consolidated from particles of decayed rocks which have been deposited from streams of water; and *metamorphic*, either igneous or sedimentary rocks which have undergone structural change due to pressure or heat. For structural purposes granite, gneiss, trap rock, limestone marble, sandstone, quartzite and slate are the most important rocks. Of these, granite and trap rock are igneous, limestone and sandstone are sedimentary, and gneiss, marble, quartzite, and slate are metamorphic rocks.

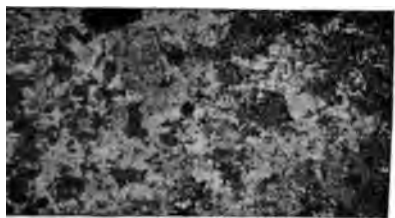


**254. Granites** are hard crystalline rocks of igneous origin which have cooled before coming to the earth's surface. True granites consist chiefly of orthoclase feldspar and quartz plus small amounts of mica or hornblende. In commerce, and among quarrymen and engineers, the term granite has a broader meaning; it includes practically all of the crystalline igneous rocks such as the syenites, which consist of feldspar plus mica or hornblende; the diorites, which are mainly hornblende plus plagioclase feldspar; the coarse-grained gabbros, which are chiefly pyroxene plus feldspar; pyroxenites, which are composed essentially of pyroxene; and peridotites, which commonly consist of olivine plus pyroxene or hornblende. Although the large majority of American stone sold for granite is correctly named from the geological standpoint, we shall use the term as applied commercially herein. For most granites the chemical constitution will fall within the following limits: Silica, 65 to 75; alumina, 12 to 18; potash, 3 to 6; soda, 2 to 5; with lime, magnesia, and the oxides of iron less than 2 per cent each.

The uses to which granite may be put are largely dependent upon the size of the crystal grains, the uniformity of the structure, the color, and durability. Only those fine-grained granites, in which the crystals are a fifth of an inch or less in diameter, are suitable for work which is to be polished or carved. The medium-grained stone may be used in building construction but the very coarse-grained rocks are, in most cases, only fit for crushed stone. Microphotographs of thin sections of granites are shown in Figs. 1a and 1b.

The uniformity of the structure of granite is often broken in several ways. *Rift* is an obscure plane, often nearly horizontal, along which the rock can be most easily split; *grain* is the plane perpendicular to the rift along which the rock splits, but with more difficulty than along the rift; and *head* or *cut-off* is the plane upon which the stone does not split. It is likely that both rift and grain were caused by strains set up in cooling from the molten state. *Joints* are fractures, produced by internal cooling strains or temperature changes, which separate the rock into sheets or be running parallel to the surface. The above-mentioned structural imperfections generally render the stone easier to quarry; although, in some cases, where the layers are very thin or the joints are badly disintegrated, they are detrimental to its use. Segregations of minerals, which cause spots or knots, and inclusions of rock fragments, are, however, defects which may render the rock worthless for ornamental purposes.

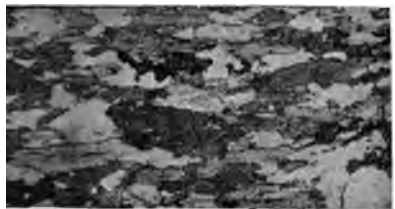
Most of the valuable granites are gray or red, although green, black and yellow stones are in use. The color of the lighter stones is largely determined by feldspar, whereas biotite, hornblende, augite, olivine and chlorite color the darker rocks. Since biotite, olivine and chlorite weather



(a) Finely Crystalline Granite.



(b) Coarsely Crystalline Granite.



(c) Hornblende Schist.



(d) Diabase, a Variety of Trap Rock.



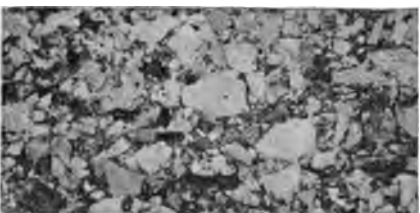
(e) Limestone



(f) Marble



(g) Very Fine Grained Sandstone.



(h) Coarse Grained Sandstone.

1.—Photomicrographs of Various Stones (Magnification = 14 diameters. Crossed nicols used on *a*, *b*, *d*, *e*, *f*, and *h*. Taken by L. W. Brown.)

*criptions* (Courtesy of Prof. A. N. Winchell.)—(*a*) Principally feldspar (several large crystals) *ca* (feathery structure).

Largely quartz (white) and feldspar (dark gray); strongly interlocked crystals.

Principally hornblende (gray), feldspar and quartz (white); some magnetite (black); inter-  
crystals.

Largely augite (gray), and plagioclase (long, black and white crystals); interlocking crystals.

Nearly all calcite; some of the grayish portions are limonite.

Nearly all calcite crystals; not well interlocked.

Chiefly quartz grains (white) with abundant magnetite (black).

Mostly quartz (white), feldspar (white with fine parallel lines), biotite and magnetite (both  
lime carbonate cement.

poorly, rocks containing them are less durable than the light-colored rocks, which are likely to be free from them.

**255. Gneiss.**—A granite which has become laminated through metamorphism is called a gneiss. Evidently there are as many varieties of gneiss as there are granites. Many of the gneisses of the Eastern states are very hard and durable; they are much used in construction but rarely for ornamentation. Gneiss in which the foliation is very fine is called a schist, Fig. 1c.

**256. Trap Rock.**—Originally trap rock was the term applied to certain igneous rocks which occurred in massive layers, or in columns that presented a stepped appearance. At the present time trap rock includes the heavy dense igneous rocks which are of dark color and which, due to rapid cooling, are very fine-grained. Basalt, diabase, the fine-grained gabbros, and occasionally fine-grained diorites, are the main types of rocks so classified.

Basalt is a black, imperfectly crystallized rock which consists mainly of augite and plagioclase feldspar. It is found quite commonly in California and the Northwestern states. Diabase (Fig. 1d), is quite similar to basalt in mineral composition, but is more perfectly crystallized. In color it varies from green to a grayish black, the green diabases being colored by olivine or chlorite. Sheets and dikes of diabase associated with red or brown sandstone are widely found in the states along the Atlantic Coast. The chief difference between gabbro and diabase lies in the kind of pyroxene, gabbro containing a foliated form. Gabbro is found in Maryland and in Minnesota. Diorite is a dark gray or greenish rock consisting of plagioclase feldspar and hornblende sometimes with a small amount of biotite. It is found in New York, Pennsylvania and Texas.

Owing to their somber colors, toughness, and the difficulty experienced in quarrying and cutting trap rocks, little use has been made of them for building construction. When crushed they make good road metal and aggregate for concrete. In California trap rock is also considerably used for paving blocks.

**257. Limestone.**—It is quite probable that most of the limestone deposits of the United States were formed from the remains of corals, molluscs and other calcareous organisms when the ocean covered these portions of the earth's surface. Limestone is the trade name for all stratified rocks, which consist principally of calcite or a combination of calcium and magnesium carbonates. Limestones which contain a considerable proportion, say 15 to 40 per cent, of magnesian carbonate are called magnesian limestones; those which contain approximately equal amounts of the two carbonates are dolomites; and those which consist principally of magnesian carbonate are termed magnesites.

In the majority of limestones the content of lime plus magnesian car-

bonates will run over 75 per cent. When the silica content is high the stone grades into a sandstone; if the clay content runs high, it approaches shale. Many limestones are contaminated with some or all of the following compounds: Clay, flint, sand, iron carbonate, iron oxide, iron sulphide, gypsum, and alkali carbonates. Of these impurities, quartz veins, flint seams and pyrite inclusions are objectionable if the rock is subjected to the weather.

Generally the grain of limestone is so fine that individual crystals are not readily distinguished by the eye (see Fig. 1e). In color, limestones vary from white to black, although the gray and blue rocks are the most abundant. Iron compounds are responsible for the pink, red and yellow varieties, while the presence of carbonaceous material is commonly the cause of the blue, gray or black color.

In addition to the varieties already mentioned several other classes of limestone are sometimes used for building construction. Among these are the famous oölitic limestone of Indiana (Bedford stone), which is composed of minute round grains cemented together; the fossiliferous limestones of Ohio; the coquina of Florida, a soft limestone which consists of a cemented aggregation of more or less broken shells; and the travertine of Italy, the calcareous constituents of which were deposited by running streams or springs of hot water. Italian travertine was used for the interior of the Pennsylvania Terminal in New York City and the old Coliseum at Rome.

**258. Marble.**—When a limestone or dolomite has become crystallized through the combined influence of great heat and pressure it is called a marble. Commercially, however, the term marble is often applied to any limestone which is capable of taking a polish; for example, the non-crystalline fossiliferous marbles of Tennessee. Besides calcite and dolomite, marbles often contain impurities, such as mica, pyrite, iron oxide, quartz, tremolite and carbon. With the possible exception of carbon, any of these impurities in segregated form is likely to cause the stone to weather non-uniformly; mica and pyrite are especially bad in this regard.

Most of the more desirable marbles are finely crystalline rocks (Fig. 1f), of white or gray color. Many of the marbles of the New England States, New York, Georgia and Alabama are of this type. However, both in texture and color, marble is subject to considerable variation. In some stones crystals  $\frac{1}{4}$  in. in diameter are in evidence; while in the ornamental but non-weathering brecciated marbles the stone body is made up of small particles of crushed rock cemented by mineral matter. The colors of marbles range from beautiful shades of pink, yellow and red to blue, brown and black. Iron compounds are generally responsible for the yellow and red shades, while carbonaceous material produces the gray, blue and black hues.

Onyx marble is another type of ornamental stone which is of some importance commercially. These translucent rocks consist of colored bands of nearly pure calcium carbonate which have been formed by the deposition of layers of colored calcareous sediments from springs and streams of cold water. California, New Mexico, Arizona, and Utah produce small quantities of onyx marbles.\*

**259. Sandstone.**—For the most part, sandstones consist of quartz grains cemented together by silica, clay, iron oxide, or lime carbonate. Feldspar, mica and pyrites are common accessory minerals. Sandstones are of sedimentary origin and have resulted from the consolidation of sand and gravel beds which either contained the cementing substance or were impregnated with it during the process of solidification.

Inasmuch as the color, hardness, strength and durability of sandstone are due largely to the binding agency, a knowledge of the character and condition of the cement is of great importance in judging of the value of this stone. Silica forms the most enduring binder, but it is likely to render the stone hard and difficult to work. Clay in combination with silica often makes a good binder; when segregated in seams it forms planes of weakness. Also since a clay cement greedily absorbs water, it probably weakens the resistance of the stone to freezing. Iron oxide is the main coloring agent and provides good bond. Probably the poorest cement is lime carbonate which, although strong, is somewhat soluble in rain water, and consequently weathers poorly.

Sandstones with a lime carbonate cement are termed calcareous; those containing an iron oxide binder are called ferruginous; and those having a clay cement are sometimes called argillaceous sandstones. *Quartzite* is a hard, metamorphic sandstone containing a silicious cement.

The textures of sandstones vary greatly. In most of the ornamental varieties the grains are less than a fiftieth of an inch in size, whereas some of the stones used for rough work contain stone particles several inches in diameter. Figs. 1*g* and 1*h* show fine and medium-grained sandstones. Sandstones containing rounded pebbles are called *conglomerates*; those having inclusions of angular stone are *breccias*.

The yellow, red and brown sandstones are generally colored by the oxides of iron; bluish and greenish shades are often caused by pyrite or iron carbonate; clay is likely to make the stone gray.

The following varieties of sandstone are of chief importance in building construction in the United States: Brownstone, a sandstone of granitic origin containing more or less feldspar and mica which is found in the Connecticut River Valley and in the Middle Atlantic States; Medina red sandstone of New York; Potsdam sandstone, a hard, red to brown rock quarried extensively in New York and to some extent in Northern

\* True onyx is a translucent silicious rock of wavy banded structure.

Wisconsin; and Berea sandstone, a finely crystalline stone of uniform texture and light yellow-gray color found in Ohio. Bluestone, a thinly bedded argillaceous sandstone found in New York and Pennsylvania, makes hard, tough flagstone. Gray Medina sandstone from New York, the Kettle River sandstone from Minnesota and the Ablemans sandstone from Wisconsin are used for paving blocks.

**260. Slate.**—When a clayey shale has been consolidated by great pressure to form a rock which may be cleaved into thin sheets, the name slate is applied. A few slates, also, have originated from the metamorphism of igneous rocks. From the compilation of a large number of analyses by Eckel \* it appears that most slates contain from 55 to 70 per cent of silica, 9 to 25 per cent of alumina, with small percentages of iron oxide, lime, magnesia, and the alkalis. The presence of iron sulphide is undesirable, since in weathering it is likely to produce rust discolorations on the surface of the slate. Slates exhibit a wide range in color from green to black. The green varieties commonly owe their color to the presence of chlorite. Most of the non-fading slates are gray or black in color.

About three-fourths of the slate produced in this country is made into roofing and the majority of the remainder is used for mill stock, blackboards, and school slates.

## THE DURABILITY OF STONE

**261. The Weathering of Structural Stone.**—This term includes the resistance of stones, when exposed to the weather, to all the disintegrating actions of heat and cold, water, frost, and chemical action, which combine in this climate to effect the rapid decomposition and destruction of most of the rocks, and of many of those which have been selected for building purposes. A stone building or monument should remain in good preservation for hundreds of years, but more commonly they begin to scale and crumble before they are twenty-five years old. The life of a rock may be many thousands of years in Egypt, or Italy, or Greece, when it would not last as many scores of years in the United States.

In quarrying and cutting the stone certain conditions arise which affect its weathering properties. Stone from the top ledges of limestone, granite and slate and from the exposed faces of the rock bed is likely to be less hard and durable than material unexposed to ground water and the weather. The method of blasting and cutting also influences the strength of the stone and its resistance to freezing and temperature changes. Small charges of powder uniformly distributed have a lesser weakening effect than large concentrations of explosives. Repeated hammering in cutting is also likely to injure the stone. Some claim saw-cut stone is more dur-

\* *Building Stones and Clays*, p. 97.

able than that finished by the hammer. Doubtless polished stone is more enduring than rough surfaced work, since the rain slides off the former more easily. Quarrying of highly absorbent stone, like the more porous sandstones, in freezing weather is bad practice, since the rock is full of quarry water and is likely to be split by freezing action. Although most stones work more readily when freshly cut and full of quarry water, yet it is unwise to lay stone in a wall immediately after quarrying. Such procedure hinders evaporation of the water and thereby lessens the strength of the stone.\* In laying stratified rock, like argillaceous sandstones, brownstones or gneisses, the natural beds should be placed horizontal in order to secure maximum weathering resistance.

Probably the most important disintegrating agent in the case of relatively impervious rocks is the variation of temperature. If one considers that most of the rocks are composed of minerals, each of which may have a different coefficient of expansion and that the crystals of a given mineral have different coefficients along their different axes, it will be appreciated that the unequal temperatures in various parts of a stone due to the way it is exposed must produce large stresses and deformations within it. Furthermore, experiments have shown that there is always some set after the temperature has been reduced (see Art. 267). Consequently, the alternate expansion and contraction due to unequal heating by the sun slowly but surely break down the structure of the stone.

The porosity of the stone and the character of its pores affect its resistance to freezing. A porous stone is less resistant to freezing than a dense stone of like pore structure and composition. It is probable that a porous stone having large pores and small tubes connecting the pores is weakened more by freezing than a stone having large pores and large connecting tubes. Also rocks with tortuous pores and tubes are more apt to be injured by freezing than those of equal porosity having straight pores and tubes.

Pyrite, magnetite, and iron carbonate oxidize in weathering and cause discoloration of the stone in which they are present. Since this oxidation is accompanied by a change in volume, the surrounding structure is weakened. When, however, pyrite is very finely and uniformly distributed through the stone, as in Berea sandstone, the structural injury due to oxidation seems to be negligible. Pyrite inclusions in limestone, or in a calcareous sandstone, are very objectionable since they may form sulphuric acid during the weathering process and the latter attacks the lime and magnesia compounds.

Inasmuch as the atmospheric water and rain of most large cities con-

\* It is claimed that the evaporation of the quarry water leaves the mineral matter which it contains in the pores of the rock, thus producing a denser and stronger surface on the stone.

tain appreciable quantities of acid, mainly carbonic and sulphuric, the solubility of the carbonates, and to a lesser extent biotite, plagioclase feldspars, and hornblende, should be remembered when selecting building stones for such localities. Limestones, marbles, the laminated micaceous brownstones, and calcareous sandstones are the greatest sufferers from the solvent action of acidulated water.

The following table prepared by Dr. A. A. Julien shows the estimated life of various building stones when exposed to weather in New York City.\*

Kind of Stone.	Life in Years
Coarse brownstone.....	5 to 15
Fine laminated brownstone.....	20 to 50
Compact brownstone.....	100 to 200
Bluestone (sandstone), untried, probably centuries.	
Nova Scotia sandstone, untried, perhaps.....	50 to 200
Ohio sandstone (best silicious variety), perhaps from one to many centuries.	
Coarse fossiliferous limestone.....	20 to 40
Fine oölitic (French) limestone.....	30 to 40
Marble, coarse dolomitic.....	40
Marble, fine dolomitic.....	60 to 80
Marble, fine.....	50 to 100
Granite.....	75 to 200
Gneiss, 50 years to many centuries.	

**262. Preservative Coatings for Stone Work.**—Various paints, oils and chemical solutions have been applied to protect stone which has begun to deteriorate. Most of these treatments, however, must be renewed in a few years. Among those which have been of some value are boiled linseed oil, paraffine, soap and alum solutions,† and Ransome's process. Before applying any of the treatments the surface of the work should be clean and dry.

Boiled linseed oil is brushed on in two or three coats and followed by a coat of dilute ammonia in warm water. The latter application unifies the discoloration produced by the oil.

Melted paraffine is sometimes applied by a brush and then forced into the pores of the stone by heating the surface to a temperature of about 60° C. In treating the surface of the Egyptian obelisk in New York City, the paraffine contained a solution of creosote and naphtha. The purpose of the creosote was to prevent organic growth on the stone.

Ransome's process consists in completely filling the pores of the stone

\* Report Tenth Census, 1880, Vol. 10, p. 391.

† For method of application of this compound, see Art. 537.



with potassium or sodium silicate. After setting until dry, this is followed by a coat of calcium chloride which must be applied with a different brush. This treatment produces a strong lime-silicate cement. Care should be taken, however, to avoid dropping either solution on painted surfaces or windows.

**263. The Value of Durability Tests.**—By far the best, and perhaps the only infallible, test of the weathering qualities of any given stone is the examination of a ledge of it which has been long exposed, or of an old building, slab or monument made from the same ledge in the quarry. Inasmuch as this test cannot be applied to a new quarry without an exposed face, and because durability is by far the most important property of any building stone, artificial tests have been formulated to supply this information.

The following tests are of some value in determining the durability of a stone of unknown weathering quality: Freezing and thawing tests, acid tests, and fire tests.

**264. Freezing Tests.**—The method ordinarily employed for the freezing test consists in immersing small cubes of stone in pure water for a period of twenty-four hours or more, then subjecting them to a temperature of 10° to 20° C. for four hours or more, and repeating this cycle twenty times or as long as necessary to produce effect upon the stone. After completion of the test the sample is dried and weighed, and the loss in weight and the appearance are noted. Owing to the time required and the small losses in weight found in this test, it is not much used. Some experimenters crush the cubes, after the final thawing but while they are still wet, and compare the strengths with results of crushing tests on unfrozen specimens. This comparison cannot be satisfactorily made unless the cubes are equally perfect and several are subjected to each test.

From a large number of tests on 2-in. cubes of Wisconsin stones in which thirty-five repetitions of freezing were used, Buckley found that granites and rhyolites \* lost less than 0.03 per cent, limestones less than 0.30 per cent and sandstones less than 0.62 per cent by weight. In crushing tests, where comparable conditions obtained, the frozen samples showed less strength than the normal specimens, some of the frozen specimens exhibiting less than 50 per cent of the strength of equally perfect normal test-pieces. Other results gotten by Bauschinger on the effect of freezing on crushing strength appear in Table 6.

Tests by Hirschwald † show that the resistance to freezing is dependent

\* A dense, fine-grained igneous rock with occasional large crystals of quartz or feldspar which has cooled near the surface of the earth and which has much the same mineral constitution as true granite; often termed quartz porphyry.

† Reported in *Handbuch der Bauweisen Gesteinsprüfung*, Hirschwald; also *Engineering Geology*, Ries and Watson, p. 454.

on the percentage of pore space occupied by water. In experiments on a number of different rocks he showed that a comparatively small number of freezings would rupture rocks which were saturated with water under pressure, whereas 25 to 30 freezings affected but little those which were immersed at atmospheric temperature and pressure. Consequently, in making the freezing test, conditions surrounding the soaking of the specimens should be uniform and the aim should be to fill the pores as much as they ever would be filled in service.

Brard's artificial test to determine the resistance to freezing has been used to some extent.\* The test consists in immersing small cubes of equal size in solution of sulphate of soda, which has been saturated at 25° C. or less, and raising the temperature to the boiling point for one-half hour. The specimens are then suspended in air for twenty-four hours to permit crystallization of the absorbed salt, sprayed with pure water from a wash-bottle, and subjected to as many repetitions of this cycle as necessary. Generally 7 to 10 cycles suffice to rupture the test-piece. Since the results of this test do not bear any fixed relation to the results of freezing tests, its value is decidedly questionable.

**265. Acid Tests.**—To ascertain the resistance of stone to acidulated waters and the acid atmosphere common to the large cities, tests have been made to determine the disintegrating effect of sulphurous and carbonic acid gases upon samples of stone. In making such tests carefully prepared cubes of equal size are dried to constant weight at 110° C. They are then placed in a large jar provided with a humidifier of some sort, and so made that it may be hermetically sealed. A stream of washed gas is then pumped through the jar until all air is expelled. The jar is then sealed for two or three days when another application of gas is administered. At the conclusion of a couple of months the test is discontinued, and the specimens are examined and weighed.

In tests on  $\frac{3}{4}$ -in. cubes of a number of stones, chiefly from New York, Wilber † found the maximum percentage losses in weight shown in Table 1.

In 44-day tests on eleven limestones of Wisconsin, Buckley found a maximum loss in weight of 1.13 per cent in sulphurous acid gas and 0.11 per cent in carbonic acid gas. His 1-in. and  $1\frac{1}{2}$ -in. cubes were badly discolored and some showed magnesium incrustations on the surface. Outside of the weight lost the effect of the carbonic acid gas was not perceptible.

Slate is sometimes tested for resistance to acidulated waters by immersing small fragments in a very dilute mixture of hydrochloric and sulphuric acids and noting the losses in weight. Merriman ‡ in his tests used 1 part

\* See Report by Luquer, *Trans. A.S.C.E.*, Vol. 33, p. 242.

† *Bulletin* 10, N. Y. State Museum, p. 357; also see *Building Stones and Clays*, by E. C. Eckel, p. 209.

‡ *Bulletin* 275, U. S. Geol. Survey.

TABLE 1.—THE EFFECT OF ACID ATMOSPHERES UPON DIFFERENT STONES. (WILBER)

Kind of Stone	No. of Tests	Maximum Per Cent Loss in Weight when Immersed in	
		CO <sub>2</sub> *	H <sub>2</sub> S†
Granite.....	4	.029	.024
Marble.....	4	.017	.250
Limestone.....	13	.087	.250
Sandstone.....	30	.104	.250
Slate.....	1	.004	.070

\* Tested 52 days. † Tested 31 days.

hydrochloric, 1 part sulphuric acid and 98 parts of water. He immersed 3×4-in. specimens in this solution and determined the dry weight before and after a 63-day immersion period. The average loss in weight of the varieties which he tested ranged from 0.286 to 0.768 grain; all but one variety had less than 0.4 grain loss.

It should be always borne in mind when making tests or comparing weight losses in corrosion tests that a knowledge of the segregated intensity of the action of the acid is often of more importance than the total loss in weight.

**266. Fire Tests.**—During conflagrations stone buildings often suffer severely. Under such conditions stone must be classed as inferior to structural clay or Portland cement products. The comparatively low resistance of building stone to high temperatures is undoubtedly due to a combination of stresses set up by the great differences in the coefficients of expansion of the constituent minerals, and also to the low heat conductivity of the rock. Quenching with a fire hose accentuates the stressed condition and often badly cracks the stone or causes thin slabs to spall off.

The relative resistances of different stones to high temperatures may be compared and the effect of structural and mineralogical differences observed by placing cubes in a muffle furnace and subjecting them to temperatures ranging from 500° to 900° C. The temperatures should be read by pyrometers so placed that the temperature of the specimens and not the flame is determined. In this connection it is well to have one or two specimens bored to take a thermo-couple, so that the inside temperature of the specimens may be estimated. When testing cubes 4 in. or less in diameter, the temperature should be held constant for at least one-half hour. The effect of variations in temperature on the strength of stone is a subject deserving of study.

For determining the resistance of stone work to fire, the only satisfactory method of testing is to build a small hut or a wall panel of stone and mortar and subject one side of the structure to high temperature,

1750° F. (954° C.), is specified by The Underwriters' Laboratories (see Art. 543).

From a considerable number of tests on small cubes reported by W. E. McCourt (*Bull.* No. 100, N. Y. State Museum), and others by E. R. Buckley (*Bull.* No. 4, Wisconsin Geol. and Nat. Hist. Survey), the following conclusions seem justifiable:

None of the common building stones, granite, gneiss, limestone, marble, or sandstone, will withstand temperatures above 850° C. Coarsely grained granite, gneiss, porous sandstones and the coarsely crystallizing marbles begin to show signs of disintegration at temperatures as low as 550° C. Limestone and marble withstand heating as well as any of the igneous rocks until the temperature is reached at which calcination begins (600 to 800° C.); then they rapidly disintegrate. Fine-grained, dense sandstone with silicious cement is very resistant to fire, but the good aspect of badly disintegrated cubes which had been heated to 850° C. indicates that the appearance of this material after a fire is misleading. Fine-grained granite at 850° C. exhibits more or less cracks but is not badly disintegrated.

Fire tests of panels of stone which were laid with broken joints in cement mortar and backed with brick or tile are reported in *Bulletin* No. 370 of the U. S. Geol. Survey. The tests were made as indicated in Art. 543. Granite, sandstone, limestone, and marble panels were tested, the stone for these panels being bought in the Chicago market. The stones composing the panels were  $4 \times 7\frac{3}{4}$  in. in cross-section. Part were laid with the narrow edge and part with the wide edge exposed. After being subjected to a temperature of 900° to 1000° C. for one hour and then soaked with a fire hose, none of the panels were in good condition. The majority of the stones in each panel were badly cracked and in some cases the faces were spalled to the depth of 2 or 3 in. No satisfactory comparison of the resistance of different classes of stone can be made on the basis of these tests.

### THE PHYSICAL PROPERTIES OF STONES

**267. The Thermal Expansion of Stone.**—The important effect of thermal expansion in the weathering of stone has already been mentioned in Art. 261. Probably the most extensive tests on the thermal expansion of American stones were those conducted at the Watertown Arsenal.\* In these experiments a large number of bars of different kinds of stone were immersed in ice-water, the temperature of which was raised to the boiling-point and again lowered to freezing. At each temperature the condition was maintained constant a sufficient time to bring the bar to

\* *Tests of Metals*, 1894 and 1895.

equilibrium while measurements were taken on a 20-in. gage length. From these tests the values of the linear coefficients of thermal expansion of Table 2 were obtained. In practically all of the stones tested a perma-

TABLE 2.—THERMAL COEFFICIENTS OF LINEAR EXPANSION FOR STONE  
(Tests of Metals, 1895)

Kind of Stone.	No. of Tests.	VALUES OF COEFFICIENT PER ° F.		
		Maximum.	Minimum.	Average.
Granite.....	12	.0000046	.0000032	.0000036
Marble.....	9	.0000063	.0000019	.0000038
Limestone.....	8	.0000047	.0000006	.0000028
Sandstone.....	13	.0000069	.0000032	.0000052

nent expansion resulted from this treatment, which averaged .00019 for granite, .00037 for marble and limestone, and .00026 in. per inch for sandstone, the variation in the amount of expansion of different stones being quite large. From results of a few of the tests, in which the stone was jacketed in sheet copper, it seems likely that a part of the permanent expansion was due to the impregnation of water into the pores of the stones during the boiling process. Nevertheless, the fact that a small permanent expansion occurs is a matter of much importance in the designing joints when very long blocks of stone are being laid.

**268. Specific Gravity and Specific Weight.**—The specific gravity is an important property for two reasons: The higher the specific gravity the greater the stability of an hydraulic structure built of the stone; also for a given kind of stone the strength increases with the specific gravity or density. The superiority of heavy stone for dam or retaining wall construction is apparent if one computes the ratio of the weights of a light and a heavy stone when submerged in water.

There are three ways of determining the specific gravity of substances like stone, all of which are in use to some extent. The first method consists in reducing the dry stone to a powder and then finding the specific gravity by the method used in testing cement (Art. 408); this method if properly applied gives the true specific gravity of the stone substance. The second method consists in drying the stone to constant weight, coating it with a thin film of paraffine, and then determining the weight of the stone immersed in water. The loss in weight divided into the dry weight gives the apparent specific gravity (i.e., the specific gravity of the stone, including its pores). The third method differs from the second in that the dried stone is saturated (?) with water and then weighed under water. Since it is impossible to completely fill the pores of the stone, the third

method gives results intermediate between those of the first and third methods. If the specific weight is computed by multiplying the specific gravity by 62.4, the results from the second method are of most importance to engineers.

From a large number of results tabulated in the *Engineering News*, Vol. 54, p. 239, by E. C. Eckel, the data in Table 3 have been abstracted. In Eckel's compilation, results are included which were obtained by all three of the above methods. Further values of specific gravity and specific weight appear in Tables 4, 5, 6 and 7.

TABLE 3.—THE SPECIFIC GRAVITY AND SPECIFIC WEIGHT OF VARIOUS CLASSES OF STONE. (ECKEL)

Stone.	No. of Varieties.	SPECIFIC GRAVITY.			SPECIFIC WEIGHT, LB.-FT. <sup>3</sup>		
		Max.	Min.	Ave.	Max.	Min.	Ave.
Trap rock.....	6	3.03	2.80	2.95	189	175	184
Granite.....	16	2.84	2.64	2.70	177	165	168
Sandstone.....	24	2.75	2.11	2.52	172	132	157
Limestone.....	7	2.76	2.48	2.64	172	155	165
Marble.....	15	2.87	2.73	2.81	179	171	175
Slate.....	4	2.84	2.75	2.79	177	172	174

**269. Porosity and Density.**—The porosity of a stone is the ratio of the volume of its pores to the entire volume of stone plus pores. Density = 1—porosity. Porosity may be accurately computed as follows: The stone, dried to constant weight, is coated with paraffine and weighed suspended in water. The loss in weight in water divided by the weight of a cubic unit of water gives the volume of stone plus pores. The volume of the stone substance may be gotten by dividing the dry weight of the stone by the product of the specific gravity of its powder and the weight of a cubic unit of water. The porosity or the density is then readily computed. Porosity is less accurately computed by the method outlined in Art. 452.

In tests on a large number of Wisconsin and Missouri stones E. R. Buckley found the following ranges in porosity for the different classes of rock: Granites, 0.019 to 1.45; limestones, 0.32 to 13.38, and sandstones, 4.81 to 28.28 per cent.\* In calculating the pore space Buckley determined the volume of water absorbed by the stone under vacuum at a temperature of 100° C.

The per cent porosity in conjunction with the per cent absorption is of value in judging of the resistance of porous materials to freezing.

\* See *Bull.* 4, Wisconsin Geol. and Nat. Hist. Survey, p. 400; and *Bull.* 2, Mo. Bureau Geol. and Mines, 2d series, p. 317.

TABLE 4.—PHYSICAL PROPERTIES OF BUILDING STONES

Condensed from Merrill's *Stones for Building and Decoration*

Kind of Stone.	Locality.	Position.	Strength per Square Inch.	Specific Gravity.	Weight per Cubic Foot.	Percentage of Absorption by Weight.	Number of Specimens averaged.
1. Granite	Grape Creek, Brownsville, Lawson, Platte Cañon, Cotopaxi, Monarch, Gunnison— <i>Colo.</i>	{ Bed Edge	Lb. 15,531 18,536	2.68	Lb. 187.3	1.1	8
2. Granite	New London, Millstone Point, Mystic River, Stony Creek— <i>Conn.</i> Vinalhaven, Fox Island, Dyer's Island, City Point, Dix Island, Jonesboro, Sprucehead, Hewitt's Island, Hurricane Island— <i>Maine.</i> Huron Island— <i>Mich.</i>	Bed	16,200	2.65	166.0	0.4	20
3. Granite	East Saint Cloud, Saint Cloud, Watab, Sauk Rapids, Beaver Bay— <i>Minn.</i>	{ Bed Edge	24,464 24,464	2.65	165.8	0.5	7
4. Granite	Cape Ann, Rockport, Quincy— <i>Mass.</i>	Bed	16,079	2.67	167.0	0.7	4
5. Granite	Fall River, Monson— <i>Mass.</i> Keene— <i>N. H.</i> Tarrytown, Morrisania, Staten Island, North River, Madison Avenue, Chaumont Bay— <i>N. Y.</i> Westerly— <i>R. I.</i> Richmond— <i>Va.</i>	Bed	15,570	2.69	168.0	0.4	14
6. Granite	New Haven— <i>Conn.</i> Duluth, Taylor's Falls, Beaver Bay— <i>Minn.</i> Jersey City Heights, Pompton— <i>N. J.</i> Goose Creek (Loudoun County)— <i>Va.</i>	{ Bed Edge	21,272 20,740	2.82	176.2	0.3	6
7. Limestone (oolitic)	Putnamville, Greensburgh, Saint Paul, Harrison County, Mount Vernon, Bloomington— <i>Ind.</i>	Bed	14,054	....	156.2	1.4	6
8. Limestone	Spencer, Ellettsville, Bedford, Salem— <i>Ind.</i>	Bed	9,297	....	145.9	3.6	8
9. Limestone	Bardstown— <i>Ky.</i>	{ Bed Edge	16,250 15,000	2.67	166.9	1.2	1
10. Limestone	Lee— <i>Mass.</i>	{ Bed Edge	22,323 21,728	.....	.....	.....	3
11. Limestone	Frontenac, Stillwater, Winona Red Wing, Kasota, Mantorville— <i>Minn.</i>	{ Bed Edge	16,320 16,643	2.52	157.3	3.1	7
12. Limestone	Glens Falls, Lake Champlain, Canajoharie, Kingston, Garrison's Station, Williamsville— <i>N. Y.</i>	{ Bed Edge	16,971 15,533	2.58	168.1	....	6

PHYSICAL PROPERTIES OF BUILDING STONES—*Continued*

Kind of Stone.	Locality.	Position.	Strength per Square Inch.	Specific Gravity.	Weight per Cubic Foot.	Percentage of Absorption by Weight.	Number of Specimens averaged.
			Lb.		Lb.		
13. Limestone (marble)	Montgomery County— <i>Pa.</i>	{ Bed	13,112	.....	.....	.....	4
		{ Edge	11,055	.....	.....	.....	
14. Limestone (marble)	Dorset— <i>Vermont.</i>	{ Bed	10,506	2.64	164.7	.....	2
		{ Edge	8,670	2.68	167.8	.....	1
15. Limestone (marble)	Italy.	Bed	12,156	2.69	168.2	.....	1
16. Sandstone	Buckhorn (Larimer Co.), Trinidad (Las Animas Co.), Manitou (El Paso Co.), Ralston, Left Hand, Saint Vairus, Fort Collins (Larimer Co.), Stout (Larimer Co.)— <i>Colo.</i> Thistle— <i>Utah.</i>	{ Bed	11,141	2.13	132.9	6.6	9
		{ Edge	12,434				
17. Sandstone	Coal Creek, Oak Creek (Fremont Co.), Gunnison (Gunnison Co.), Manitou (El Paso Co.), La Porte (Larimer Co.), Brandford (Fremont Co.)— <i>Colo.</i>	{ Bed	5,481	2.12	133.0	13.8	9
		{ Edge	4,941				
18. Sandstone	Middletown, Portland— <i>Conn.</i> East Long Meadow— <i>Mass.</i> Marquette— <i>Mich.</i>	Bed	6,639	2.27	142.2	3.5	3
19. Sandstone	Hinckley, Fort Snelling— <i>Minn.</i>	{ Bed	16,625	2.38	139.0	6.0	2
		{ Edge	18,750				
20. Sandstone	Dresbach, Jordan, Fond du Lac, Dakota— <i>Minn.</i>	{ Bed	5,789	19.9	124.4	9.9	6
		{ Edge	4,102				
21. Sandstone	Taylor's Falls, Kasota, Frontenac— <i>Minn.</i>	{ Bed	7,483	2.42	142.4	5.9	3
		{ Edge	9,725				
22. Sandstone	Haverstraw, Hudson River, Albion— <i>N. Y.</i>	{ Bed	8,925	2.78	142.2	2.6	2
		{ Edge	7,687				
23. Sandstone	Medina— <i>N. Y.</i>	{ Bed	17,500	2.42	150.8	1.6	2
		{ Edge	14,812	2.39	149.3	2.0	1
24. Sandstone	Vermilion— <i>Ohio</i>	{ Bed	7,840	2.16	135.0	5.2	5
		{ Edge	6,875				1
25. Sandstone	Seneca— <i>Ohio.</i>	{ Bed	9,687	2.39	149.3	3.1	1
		{ Edge	10,500				
26. Sandstone	Cleveland— <i>Ohio.</i>	{ Bed	6,800	2.24	140.0	2.8	1
		{ Edge	7,910				
27. Sandstone	Marblehead— <i>Ohio.</i>	{ Bed	7,937	2.11	144.4	5.2	1
		{ Edge	6,850				
28. Sandstone	North Amherst— <i>Ohio.</i>	{ Bed	6,850	2.17	144.4	5.2	2
		{ Edge	6,850				1
29. Sandstone	Berea— <i>Ohio</i>	Bed					



**270. Absorption.**—Methods for determining the per cent absorption of stone are similar to those outlined in Art. 291. If the per cent water absorbed by volume is desired, it may be gotten by multiplying the per cent by weight by the specific gravity of the rock.

The proportion of the pore space filled with water and the rate at which the rock will expel absorbed water are important criteria of resistance to freezing. Rocks which absorb enough water to fill the pores and which expel slowly are very likely to be weakened by freezing.

Values of the percentage of absorption for different stones may be found in Table 4.

### THE MECHANICAL PROPERTIES OF STONE

**271. The Strength of Stone.**—The compressive strength of building stone is the most commonly tested mechanical property; although shearing and transverse tests are sometimes made.\* The preparation of specimens and the methods of performing these tests have been considered in Ch. III. In considering the test results which follow, it must be recognized that many of the values for American stones are subject to variation owing to the uncertainty which exists surrounding the preparation of the specimens which were tested.

In masonry construction the greatest crushing load allowed on the best grade of granite ashlar does not run over 600 lb. per square inch and for the ordinary grades of coarse rubble laid in cement mortar the allowed stress generally runs between 150 and 200 lb. per square inch. Therefore, if a factor of safety of ten is allowed, the required compressive strength of stone in cubical specimens need not exceed 6000 lb. per square inch for the most severe loading. Although there is no objection to greater strength, it cannot be argued that because one stone has a crushing strength of 20,000 lb. per square inch and another 30,000 lb. per square inch that the latter is superior to the former for building purposes. A high crushing strength alone is not of great importance. Crushing strengths of the more important stones of the United States are given in Tables 4 and 5.

Owing to faulty methods of quarrying, discrepancies between the rift and planes of bedment in the wall, and owing to irregularities in foundations and mortar bedments stone is much more likely to crack due to the imposition of bending stresses than by crushing. Cracks in lintels due to transverse stresses and in walls due to excessive shear stresses are quite common. Consequently, the resistance of stone to these stresses is of importance.

\* The tests made on crushed stone for paving purposes are described in *Bull. No. 44* of the U. S. Dept. of Agriculture; also see Blanchard and Drowne's *Text Book on Highway Engineering*, Ch. 9.

TABLE 5.—TESTS OF AMERICAN BUILDING STONE MADE AT THE WATERTOWN ARSENAL

(Rep. 1894.)

Name of Stone.	Weight per Cubic Foot.	Compression Tests.		Ratio of Lateral Expansion to Longitudinal Compression.*	Shearing Strength.	Coefficient of Expansion in Water per °F.
		Strength in Pounds per Square Inch.	Modulus of Elasticity for Working Loads.			
	Lb.		Lb./In. <sup>2</sup>		Lb.	
Sedford granite (Conn.)...	162.0	15,707	8,333,300	0.250	1833	.00000398
Ord granite (Mass.).....	162.5	23,775	6,663,000	0.172	2554	.00000418
Ord granite (Mass.).....						.00000415
Ord granite (N. H.).....	164.7	26,174	4,545,400	0.196	2214	.00000337
Ord pink granite (Mass.)...	161.9	18,988	5,128,000		1825	
On Hill granite (Mass.)...	161.5	19,670	6,666,700		1550	
Le marble (Georgia).....	170.0	13,466	6,896,500	0.345	1369	
Okeee marble (Georgia)...	167.8	12,618	9,090,900	0.270	1237	.00000441
Rah marble (Georgia).....	163.8	14,052	7,843,100	0.278	1411	
Seesaw marble (Georgia)...	168.1	9,562	7,547,100	0.256	1242	
Marble (Mass.).....						.00000454
Blue Hill marble (Ga.)...	168.6	11,505	9,090,900	0.294	1332	.00000194
Tahoe marble (N. Y.).....	178.0	16,203	13,563,200	0.222	1490	.00000441
Vernon limestone (Ky.)...	139.1	7,647	3,200,200	0.250	1705	.00000464
Ord blue limestone (O.)...		10,823	7,250,000	0.270	1017	.00000389
Sh River bluestone (N. Y.)...		22,947	5,268,800			
Son slate (Maine).....						.00000519
Per sandstone (Oregon)...	159.8	15,163	2,816,900	0.091	1831	.00000177
Lstone, Cromwell (Conn.)...		10,780				
Nard sandstone (Mass.)...	133.5	9,880	1,941,700	0.333	1204	.00000567
Se sandstone (Mass.)...	133.4	10,363	1,834,900	0.300	1150	.00000577
Seester sandstone (Mass.)...	136.6	9,762	2,439,000	0.227	1242	.00000517
Mac sandstone (Md.).....						.00000500
Spia sandstone (Oregon)...		12,665				.00000320
Kanuk sandstone (Wash.)...		11,389			1352	
Terhoff Portland cement, at.....						.00000578

\* Poisson's ratio.

In Table 5 are given the shearing strengths for a number of American stones. Inasmuch as the majority of these tests were made by a method similar to that outlined in Art. 14 (Fig. 11, Ch. XIV), the values are probably lower than the true shearing strength due to bending. While finding the transverse strength of Wisconsin stones, Buckley found the following ranges in modulus of rupture: Granite, 2713 to 3910; limestone, 1164 to 4659, and sandstone, 363 to 1324 lb. per square inch. In Table 6 is presented a series of tests on Bavarian building stones. These tests were made with great care and precision by Prof. Schinger and reported in his *Communications*, Vol. 10. The results show that there is no fixed relation between the various kinds of strength and stone.

A considerable number of transverse tests were made by Prof. Merri-

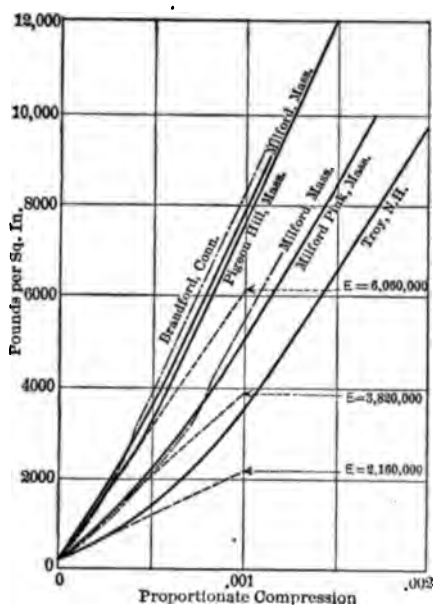


FIG. 2.—The Elastic Properties of Various Granites under Compressive Stress. (*Wat. Ars. Rept.*, 1894.)

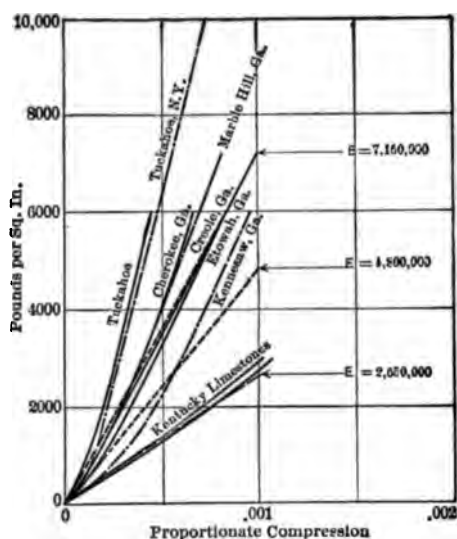


FIG. 3.—The Elastic Properties of Various Limestones and Marbles under Compressive Stress. (*Wat. Ars. Rept.*, 1894.)

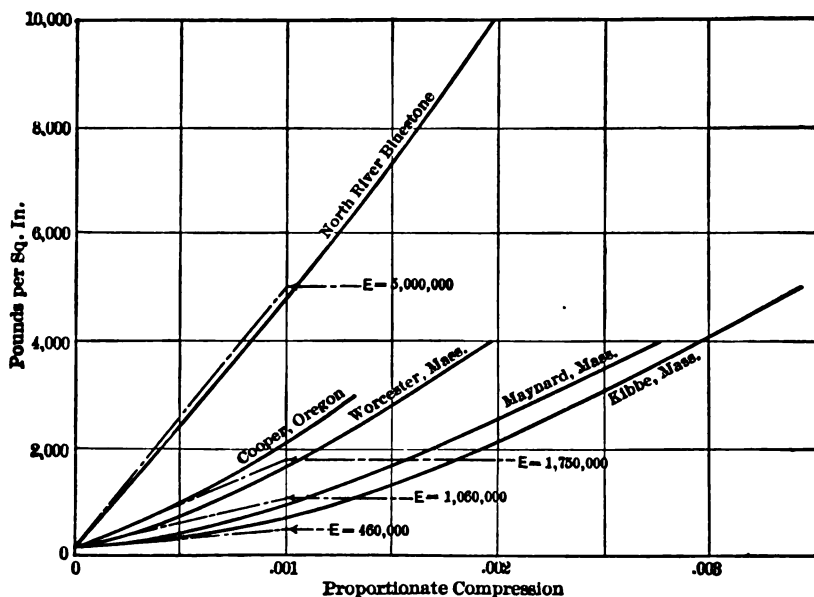


FIG. 4.—The Elastic Properties of Various Sandstones under Compressive Stress. (*Tests of Metals*, 1894.)

man \* on slate specimens which were secured from a number of the Eastern states. The specimens were  $24 \times 12$  in. in plan and  $\frac{3}{8}$  to  $\frac{1}{2}$  in. thick. They were tested flatwise on a 22-in. span. The values of the average modulus of rupture for nine varieties ranged from 6410 to 9880 lb. per square inch, and the average maximum deflection varied from 0.19 to 0.23 in. It seems reasonable, therefore, to expect good slate to have a modulus of rupture of 7000 lb. per square inch and to deflect 0.20 in. at rupture.

Experiments made at the Watertown Arsenal † have shown that a

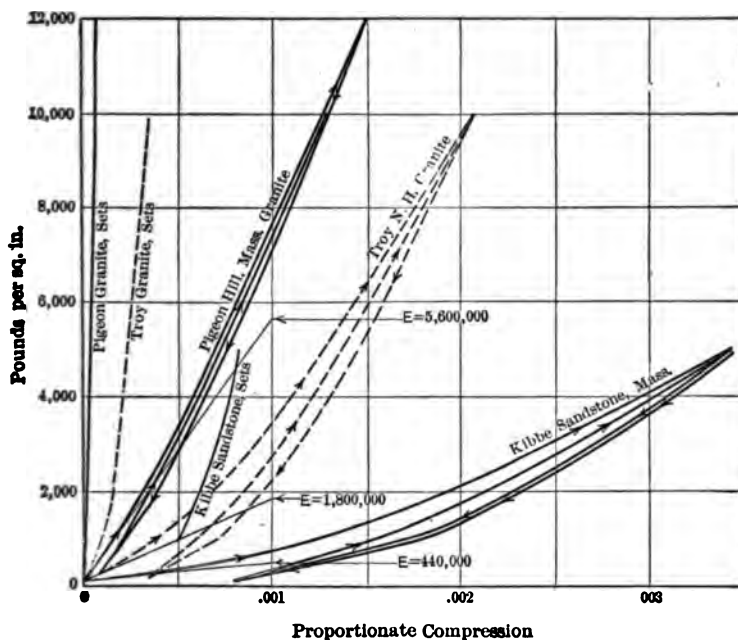


FIG. 5.—Elastic Properties of Various Stones under Compressive Stress. (*Tests of Metals*, 1894.)

very great loss in transverse strength results from immersing stone in hot and cold water. The granites were the least affected and the marbles most. The loss in strength of the former, for the most part, was less than 25 per cent while several of the marbles lost 50 per cent of their strength due to this treatment. Consequently, one must be careful that specimens of stone for transverse tests are not subjected to large temperature variations. Furthermore, this fact should be borne in mind in designing lintels and stone beams which will be subjected to wide variations of temperature.

**272. The Elastic Properties of Stone.**—Like cast iron, brick and

\* Bull. No. 275, U. S. Geol. Survey.

† *Tests of Metals*, 1905.

concrete, stone is a material which does not obey Hooke's law. The granites, limestones and marbles, however, exhibit less curvature and less set in their stress-deformation curves than do the more porous sandstones. These facts will be evident after an examination of Figs. 2, 3, 4 and 5. Bauschinger has shown, however, that for a given specimen the moduli of elasticity in tension, compression and in cross-bending are practically the same. The values which are taken from his tests, Table 6, were found on the first loading.

TABLE 6.—PROPERTIES OF THE BUILDING STONES OF BAVARIA.

(Bauschinger's *Communications*, Vol. 10, 1884)

Strengths given in Pounds per Square Inch.

Kind of Stone.	Specific gravity.	Weight per Cubic Foot.	Cross-bending.		Compressive Strength.			Tensile Strength.	Shearing Strength.	
			Modulus of Elasticity.	Modulus of Rupture.	Perpendicular to Bed.	Parallel to Bed.	Parallel to Bed after 25 Freezings.		Perpendicular to Bed.	Parallel to Bed.
Granite.....	2.65	165.4	2,986,000	1365	19,200	18,910	21,470	619	1379	142
Granite.....	2.66	166	1,621,000	1194	19,200	20,050	20,480	683	1450	853
Triassic limestone.....	2.48	154.8	6,420,000	882	8,130	8,320	6,810	583	555	384
Jurassic limestone.....	2.23	139.2	.....	.....	11,110	7,410	12,290	448	739	540
(marble).....	2.08	129.8	4,906,000	462	4,664	8,760	3,313	213	498	299
Oolitic limestone.....	2.72	169.7	.....	1792	19,340	20,620	18,770	910	1479	1138
Tuffa stone.....	1.80	112.3	.....	469	1,195	2,545	2,076	227	227	213
Variegated sandstone.....	2.06	128.5	426,600	469	7,420	6,010	6,730	107	569	355
Variegated sandstone.....	2.20	137.3	867,400	718	9,040	7,790	7,910	199	512	313
Variegated sandstone.....	2.28	142.3	1,340,000	1109	12,930	13,410	11,520	576	910	540
Variegated sandstone.....	2.00	124.8	341,300	341	6,160	6,100	4,877	128	455	427
Carboniferous sandstone.....	2.20	137.3	910,000	483	7,636	8,390	5,986	341	640	284
Carboniferous limestone.....	2.23	139.1	334,200	441	6,684	6,670	5,900	213	583	469
Slaty sandstone.....	1.82	113.6	512,000	249	3,071	2,247	2,161	98	370	242
Slaty sandstone.....	1.92	119.8	270,200	135	3,029	2,659	4,252	67	242	185
Green sandstone.....	2.15	134.2	583,000	156	4,707	4,308	4,038	94	341	327
Cretaceous sandstone.....	2.60	162.3	568,800	597	13,510	14,500	.....	327	668	370
Cretaceous sandstone.....	2.73	170.4	2,687,000	967	28,860	17,490	.....	512	995	768
Quartz conglomerate.....	2.29	142.9	1,763,000	654	5,546	4,408	3,270	242	.....	.....

In Table 5 appear values of Poisson's ratio for stone. With one or two exceptions the values are about the same as those given for the ferrous metals; one-fourth is a fair average.

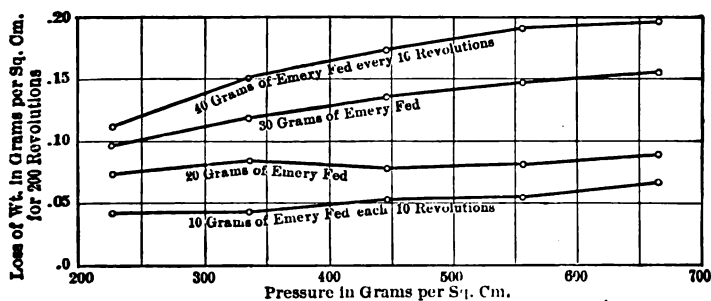
**273. Resistance to Abrasion.**—The abrasion of traffic on pavements, sidewalks and doorsteps is a matter of considerable importance in determining the life of stone used for such purposes. A number of tests have been devised for measuring abrasive resistance, but none have been universally adopted. Two types of test have been considerably used. In one a carefully prepared stone specimen with a plane face is held against

horizontal table which revolves about a vertical axis, and abrasion is caused by sand or emery. In the other test the specimen is subjected to the action of a sand blast under a standard pressure.

Bauschinger experimented considerably with the first method, using a circular iron table 5 ft. in diameter. He placed two specimens each 4 in. in diameter at a distance of 19.5 in. from the axis and weighted each with 30 lbs. The table was run at a speed of 20 r.p.m. and 20 grams of emery (No. 3) was fed to the plate every 10 revolutions, the old emery was brushed off. Two attendants constantly kept the emery in the path of the specimen.\*

The results of some of Bauschinger's preliminary tests to determine the effect of pressure and rate of feeding the emery are shown in Fig. 6. Table 7 gives the average results which he obtained in testing various materials. The results indicate:

That the wet grinding was about twice as effective as the dry grind-



—Showing the Relation between the Abrasion, Pressure, and Energy Used in Abrasion Test. (Bauschinger.)

The exact average ratios being given in the last column of the table for various species of stone.†

There is no fixed relation between crushing strength and abrasive resistance.

The limestones wear about five times and the sandstones about four times as fast as the granites, porphyries, and basalts.

The clay-slate shows the best results in abrasion, but only a few specimens were tested.

Bauschinger's test does not differ greatly in principle from the Dorry hardness test used in France and the United States for road metal. In the Dorry test the abrasive is standard quartz sand passing a 30-mesh and retained on a 40-mesh sieve; diameter of each specimen is 1 inch; and the speed is 30 r.p.m. for 1000 revolutions. These ratios have been taken from the wet and dry tests on identical material, therefore are not the ratios of the two general average results in the previous section.

TABLE 7.—AVERAGE RESULTS OF BAUSCHINGER'S ABRASION TESTS OF PAVING MATERIAL

*(Communications, Vol. 11, 1884)*

Four-inch cubes of the material were pressed on an iron plate with a weight of 4 pounds per square inch, and 20 grams of emery fed every 10 revolutions. Results obtained for 200 revolutions at a radius of 19.5 inches.

Kind of Material.	Average Specific Gravity.	Average Weight per Cubic Foot in Pounds.	Average Compressive Strength in Pounds per Sq.in. Inch.	Number of Results Averaged.	How ground: Dry or Wet.	Average Loss of Volume in Cubic Inches.	Ratio: Loss wet Loss dry
Granite.....	2.63	164	22,400	{ 92 8	dry wet	0.24 0.46	} 1.72
Syenite.....	2.27	142	18,780	{ 24 1	dry wet	0.28 0.82	} 1.90
Diorite.....	2.87	180	26,200	{ 18 2	dry wet	0.27 0.68	} 1.90
Hornblende.....	2.82	176	21,900	2	dry	0.19	
Porphyry.....	2.57	161	24,500	{ 93 8	dry wet	0.20 0.24	} 1.72
Basalt.....	3.01	188	34,200	{ 86 4	dry wet	0.19 0.47	} 2.31
Gneiss.....	2.61	163	23,000	4	dry	0.21	
Quartz.....	2.63	165	17,500	9	wet	0.19	
Clay-slate.....	2.72	170	26,000	{ 8 2	dry wet	0.16 0.35	} 2.76
Breccia.....	2.61	163	22,600	10	dry	0.20	
Limestone.....	2.87	180	20,500	{ 163 32	dry wet	1.10 1.41	} 1.60
Sandstone.....	2.48	155	17,600	{ 44 38	dry wet	0.81 0.64	} 2.25
Brick and tile....	2.98	187	.....	{ 105 34	dry wet	0.38 0.75	} 2.50
Artificial stone } made with Port- land cement.... }	2.36	148	.....	{ 20 4	dry wet	0.51 1.82	} 3.20
Asphalt paving....	2.33	146	.....	{ 2 2	dry wet	0.61 1.62	} 2.68

5. The brick and tile wear about twice as fast and the cement compositions about three times as fast as the primitive rocks.

5. The resistance of asphalt paving to abrasion falls between the cement mixtures and sandstone.

Prof. M. Gary, of the Royal Testing Laboratory at Berlin, has used the sand-blast method of testing considerably and has reported comparisons of the grinding table and sand-blast methods. Gary used a special sand-blasting device fitted with a nozzle 6 cm. in diameter. The sand was pro-

led by a dry-steam pressure of 3 atmospheres for a two-minute period actly against the stone. In testing with the table he used specimens 50 sq. cm. (7.75 sq. in.) in area and placed them at 32 cm. (9.6 in.) from the axis of the table. Results obtained by both methods on similar stones are given in Table 8.

TABLE 8.—RESULTS OF ABRASION TESTS ON BUILDING STONES

(Gary in *Baumaterialienkunde*, Vol. 10, p. 136)

Area of specimens used on grinding table = 7.75 sq. in.

Diameter of nozzle on sand-blasting device = 2.36 in.

Kind of Stone.	Compressive Strength Lb./In. <sup>2</sup>	VOLUME OF WEAR IN CUBIC INCHES PER SQUARE INCH.		
		On Grinding Table.	With Sand Blast.	
			Perpendicular to Rift.	Parallel to Rift.
Basalt . . . . .	38,900	0.042	0.024	0.025
Granite . . . . .	21,360	0.041	0.037	0.052
Gneiss . . . . .	21,230	0.079	0.056	0.045
Porphyry . . . . .	17,840	0.068	0.046	0.036
Graywacke *. . . . .	15,780	0.085	0.059	0.058
Sandstone . . . . .	6,640	0.144	0.155	0.117
Slate . . . . .	7,480	0.234	0.111	0.082

\* A dense sandstone containing rounded or angular particles of quartz, feldspar or slate.

Although the sand-blast test brings out the weak spots in the specimen, it is questionable whether it approximates the action of traffic on the stone. On the other hand, when using the grinding table, considerable trouble is experienced in maintaining the abrasive agent in standard condition and in forcing it under the specimen in a uniform manner.



## CHAPTER VIII

### STRUCTURAL CLAY PRODUCTS \*

**274. Introduction.**—Clay products form one of the most important classes of structural materials. In building construction, brick and terra-cotta are desirable on account of their pleasing appearance, strength and durability. Partition and floor tile form walls and floors of light weight which possess high strength and resistance to fire. Paving brick make economical and durable, although somewhat noisy, pavements. Clay pipe on account of their durability, strength, light weight, and cheapness are successfully used in sewers, drains and conduits.

Structural clay products may be classified as follows:

Brick .....	{	Building brick
		Paving brick
		Fire brick
Building tile.....	{	Hollow blocks
		Partition tile
		Fireproofing
		Roofing tile
		Floor tile
		Wall tile
Terra-cotta.....	{	Decorative terra-cotta
		Terra-cotta lumber
Pipe.....	{	Sewer pipe
		Drain pipe
		Conduit pipe

The total value of the clay products produced in the United States, in 1914, was \$164,986,983. Of this amount the value of common brick was 27 per cent; fire-brick 10.0 per cent; building tile 8.5 per cent; sewer pipe, 8.5 per cent; vitrified brick, 7.6 per cent; front brick, 5.6 per cent; drain tile, 5.2 per cent, and architectural terra-cotta, 3.7 per cent.

\* The following texts have been freely consulted in preparing this chapter: *Clays: Occurrence, Properties and Uses*; also *Building Stones and Clay Products*, by H. Ries; Wiley & Sons. *The Clay Workers Handbook*, by A. B. Searle; Griffin & Co. *Modern Brick Making*, by A. B. Searle; Scott, Greenwood & Son.

## MATERIALS, MANUFACTURE AND TESTING OF CLAY PRODUCTS

## THE RAW MATERIALS

**275. Classes of Raw Materials.**—In addition to the various types of clay there are also many forms of shale which are used in the production of clay products. Clays are those substances resulting from the decay of rocks, which possess plasticity on being tempered with water and which are capable of retaining their shape when molded into various forms and dried. When such bodies are heated to redness or above they resemble stone in hardness and strength. Shale is a hardened form of clay which has been consolidated by the weight of overlying earth, but which after being reduced to a powder exhibits the above-mentioned characteristics.

**1. Local clays** are formed from the decay of the underlying rocks. They constitute important sources of high-grade clays for pottery. Those clays which have been removed from the parent rock by glacial action, by water or by wind are called transported clays. Such clays are often termed sedimentary since they have been carried as sediment by the current and deposited in places where the velocity of flow decreased. On account of the changes in conditions which surrounded the deposition of sedimentary clays, they generally consist of strata of material which often vary considerably in composition and properties. Frequently such clays have sandy laminations or are mixed with sand.

Since the largest and most homogeneous deposits of sedimentary clay are those precipitated in large bodies of still water; the marine clays, deposited on former ocean bottoms, or lacustrine clays, found on the bottoms of extinct lakes or in swamps, form the most valuable sources of raw materials for the manufacture of structural clay products. The soft clays, either glacial or residual, which are found at or near the surface are often termed surface clays. Fire-clay is a term, loosely applied, to include those sedimentary or residual clays which vitrify at a very high temperature and which, when so burned, possess great resistance to heat. Impure fire-clays are contaminated with certain fluxes such as lime, iron oxide or the alkalis which reduce the vitrification temperature. In many of the coal-producing states fire-clays underlie the coal beds.

**276. Composition of Clays.**—In determining the suitability of clays for the manufacture of clay products a knowledge of both the mineral and chemical constitution is of assistance. From the mineral constitution the proportion of true clay substance may be gotten, whereas the chemical constitution affords indications of the purity, refractoriness, color, and shrinkage or swelling in burning.

The minerals most commonly found in clays and shales are kaolinite ( $2\text{SiO}_2, \text{Al}_2\text{O}_3, 2\text{H}_2\text{O}$ ), and other hydrated silicates of alumina; quartz

( $\text{SiO}_2$ ); feldspar (principally silicate of alumina combined with potash, or lime, or soda and lime); limonite ( $2\text{Fe}_2\text{O}_3, 3\text{H}_2\text{O}$ ); hematite ( $\text{Fe}_2\text{O}_3$ ); siderite ( $\text{FeCO}_3$ ); pyrite ( $\text{FeS}_2$ ); calcite ( $\text{CaCO}_3$ ); magnesite ( $\text{MgCO}_3$ ); gypsum ( $\text{CaSO}_4, 2\text{H}_2\text{O}$ ), and sometimes rutile ( $\text{TiO}_2$ ). Of these minerals, kaolinite and other hydrated silicates of alumina are the most desirable constituents. They generally form the major part of those high-grade clays, termed kaolins, which are used in the production of crockery and white burning pottery. These silicates constitute the finer portion of the clay which is called the clay substance. By Seger, clay substance is defined as the material less than 0.004 in. in diameter.

Chemical analyses of good clays will show that they consist mainly of the following elements:

Silica	( $\text{SiO}_2$ )	} Fluxing Ingredients (generally less than 20 per cent)
Alumina	( $\text{Al}_2\text{O}_3$ )	
Ferric Oxide	( $\text{Fe}_2\text{O}_3$ )	
Lime	( $\text{CaO}$ )	
Magnesia	( $\text{MgO}$ )	
Alkalies	( $\text{K}_2\text{O} + \text{Na}_2\text{O}$ )	
Water	( $\text{H}_2\text{O}$ )	
Carbon Dioxide	( $\text{CO}_2$ )	
Sulphur Trioxide	( $\text{SO}_3$ )	

Silica generally forms from 40 to 80 per cent of the raw materials used in making structural clay products other than fire brick. In the latter the silica content may rise to 98 per cent. Although a large percentage of sand or uncombined silica in clay is undesirable, it is sometimes added to decrease shrinkage in burning and to increase the refractoriness of low alumina clays.

The alumina content ordinarily ranges from 10 to 40 per cent except in silica brick. Wares having an exceedingly high alumina content are likely to be very refractory.

Iron oxide, which in most cases constitutes less than 7 per cent of a clay, is a most important factor in determining the color of the clay and the burned product. It also tends to lower the fusion point of the clay, especially if present as ferrous oxide.

Lime normally constitutes less than 10 per cent of clay, but in some glacial deposits, which are successfully used in making common brick and tiling, a higher lime content obtains. In carbonated form lime lowers the fusion point. Since the carbonate breaks up into carbon dioxide ( $\text{CO}_2$ ) and lime ( $\text{CaO}$ ) at a temperature of  $900^\circ\text{C}$ .; it is desirable, in clays burned at this temperature, to finely crush the lime pebbles. If this is done, danger from "popping" in the burnt ware, due to slaking of the lime, may be avoided. If the burning temperature is considerably higher than the

above, complex combinations of lime, silica, and alumina are formed with the result that the lime effects a change in the color of the product. Red-burning wares are often made buff-burning by increasing the lime content.

Magnesia rarely exceeds 1 per cent in clay. In burning it causes the clay to soften at a slower rate than does lime and lessens warping.

The alkalis, forming less than 10 per cent of the raw clay, are of great value as fluxes, especially when combined with silicates of alumina. Feldspar is much used as a flux with kaolin in making white ware.

A large proportion of free water generally causes clay to shrink considerably in drying; combined water causes shrinkage in burning.

Carbonaceous material in the form of bituminous matter or carbon greatly affects the color of the raw clay. Unless proper precaution is taken to effect complete removal of such matter by oxidation, the burned product is likely to have a black core.

Sulphur is most commonly found in clay as the sulphate of calcium, cesium, potassium, sodium or iron, or as iron sulphide. Generally the portion is small. If, however, there is carbon in the clay and insufficient time is given during burning for proper oxidation of carbon and sulphur, the latter will cause a spongy swollen structure in the burned product. Most of the sulphates are soluble and give evidence of their presence by the formation of a scum on the dried ware. As a result, unless careful care is exercised in burning, the product will be discolored by white blotches. The use of water containing small quantities of magnesium or calcium carbonates, together with a sulphurous fuel often causes similar effects. Wall white, which appears after brick have been laid, may be due to soluble salts in the brick or in the mortar, which are brought to the surface by absorption of water and subsequent drying.

**277. Physical Properties of Clays.**—Plasticity, tensile strength, texture, shrinkage, porosity, fusibility, and color after burning are the physical properties which are of most importance in determining the value of a clay. A knowledge of these properties is of more benefit in judging the quality of the raw material than a chemical analysis.

By plasticity is meant the property which wetted clay has of being permanently deformed without cracking. The amount of water required by different clays to produce the most plastic condition varies from 15 to 35 per cent. Although plasticity is probably the most important physical property of clay, yet there are no methods of measuring it which are entirely satisfactory.\* The simplest and most used test is afforded by feeling of the wetted clay with the fingers. Personal equation necessarily plays a large part in such determination.

Since clay ware is subjected to considerable stress in molding, handling,

\* For a more complete discussion of the physical properties and their measurement, see *Clays their Occurrence and Uses*, by H. Ries, Ch. 3.

and drying a high tensile strength is desirable. The test is made by determining the strength of specimens which have been molded into briquette form \* and very carefully dried. The tensile strength of clays will vary from almost nothing in highly silicious fire clays to over 400 lb. per square inch in some common brick clays. Kaolin generally shows low tensile strength.

The texture of a clay is measured by the fineness of its grains. In rough work the per cent passing a No. 100 sieve is determined, but for measuring the size of the clay grains a more refined device such as the Schöne washing apparatus or a centrifugal separator is used. No numerical limit to the grain size or desired relation between sizes has been established. Tests by Beyers and Williams † indicate that the sizes of grain from 0.004 in. down should be uniformly graded to obtain maximum tensile strength. Ries' tests showed that an excess of either very fine material or of sand grains decreased the tensile strength of the clay. Very fine-grained clays free from sand are more plastic and shrink more than those containing coarser material.

A knowledge of the shrinkage both in drying and in burning is required in order to produce a product of required size. Also the amount of shrinkage forms an index of the degree of burning. The shrinkage in drying is dependent upon pore space within the clay and upon the amount of mixing water. The addition of sand or ground burnt clay lowers shrinkage, increases porosity and facilitates drying. Fire-shrinkage is dependent upon the proportion of volatile elements, upon texture and the way that clay burns. Tests of shrinkage are made by determining the volume of benzine displaced by a small prism of clay when green, after drying at a temperature slightly above the boiling point of water, and also after burning. Beyer and Williams, in tests on a number of Iowa clays, reported air-shrinkages varying from 4.86 per cent to 27.00 per cent and fire-shrinkages from -2.88 per cent (swelling) to 5.92 per cent.

By porosity of clay is meant the ratio of the volume of pore space to the dry volume. Since porosity affects the proportion of water required to make clay plastic, it will indirectly influence air-shrinkage. Large pores allow the water to evaporate more easily and consequently permit a higher rate of drying than do small pores. Inasmuch as the rate at which the clay may be safely dried is of great importance in manufacturing clay products, the effect of porosity on the rate of drying should be considered.

The temperature at which a clay fuses is determined by the proportion of fluxes, texture, homogeneity of the material, character of the flame, and its mineral constitution. Owing to non-uniformity in composition, parts of the clay body melt at different rates so that the softening period extends

\* See briquette molds and testing machines in Ch. XII.

† *Iowa Geol. Surv.*, Vol. 15, p. 102, 1904.

over a considerable range both of time and temperature. Wheeler divides the period into (1) incipient vitrification, at which the clay has softened sufficiently to cause adherence but not enough to close the pores or cause loss of shape—on cooling the material cannot be scratched by the knife; (2) complete vitrification, more or less well-marked by maximum shrinkage, coalescence of particles, smooth fracture and no loss in shape; (3) viscous vitrification, produced by a further increase in temperature which results in a soft molten mass, a gradual loss in shape, and a glassy fracture after cooling.

Experiments roughly indicate that the higher the proportion of fluxes the lower the melting-point. Fine-textured clays fuse more easily than those of coarser texture and the same mineral composition. The uniformity of the clay mass determines very largely the influence of various elements; the carbonate of lime in large lumps may cause popping when present in small percentages, but when finely ground 15 per cent of it may be allowed in making brick or tile. Lime combined with silicate of alumina (feldspar) forms a desirable flux. Iron in the ferrous form, found in carbonates and in magnetite, fuses more easily than when present as ferric iron. If the kiln atmosphere is insufficiently oxidizing in character during the early stages of burning, the removal of carbon and sulphur will be prevented until the mass has shrunk to such an extent as to prevent their expulsion and the oxidation of iron. When this happens a product with a discolored core or swollen body is likely to result.

Since a determination of the fusibility of a clay is of much importance both in judging of the cost of burning it and in estimating its refractoriness, experiments are often made on small prisms to determine the rapidity with which the clay may be burned, the temperatures at which incipient, complete and viscous vitrification occur, how the clay behaves in annealing and the color of the burned product. Temperatures are commonly measured by means of Seger cones,\* in refined work by a pyrometer.

#### METHOD OF MANUFACTURE

**278. Preparation of the Clay.**—Many of the large deposits of clay or soft shale are worked in open cut with a steam shovel. The hard shales adjacent to coal veins are frequently mined. Generally the raw material is drawn from the pit in cars on a narrow-gage track by horses or by dinky engines. For pressed brick and terra-cotta it is sometimes advantageous to weather the clay, before using it. This is accomplished by loosely spreading the clay in a layer a couple of feet thick over a flat surface where

\* Seger cones are made from mixtures of clay and fluxes so proportioned that their melting-points form a temperature scale. Two or more cones differing in fusibility are inserted in the furnace or kiln and the temperature estimated from their appearances.

it will be exposed to the action of the elements. Such action causes a rusting of iron particles and a breaking down of pyrite inclusions.

Most clays, however, are hauled directly to a crushing or disintegrating device. For the hard shales jaw-crushers are sometimes used. Dry pans like the one shown in Fig. 1 are often employed to break up the softer

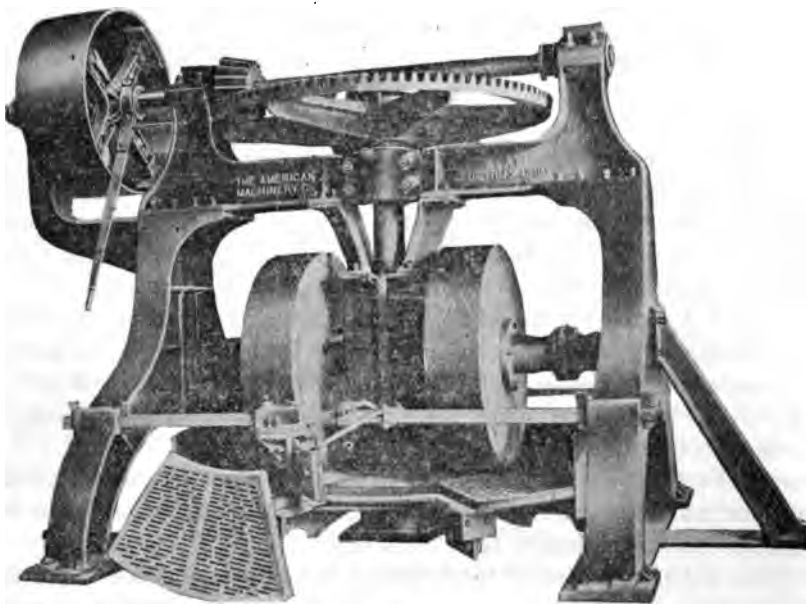


FIG. 1.—A 9-ft. Dry Pan.



FIG. 2.—A Double-shaft Pug Mill.

shales and tough clays. For certain dry shales, toothed or corrugated rolls are effective where coarse grinding only is required. After the clay has been crushed it is conveyed to a pug-mill (Fig. 2) in which it is tempered with the proper amount of water and thoroughly mixed. Wet clays are generally ground and tempered simultaneously in large ring pits, in pug-mills or in wet pans. In the ring pits a heavy iron wheel which rolls over the bottom surface of the pit in a spiral path, serves to stir and grind the

clay. This method is used only at small plants. The wet pan is much like the dry pan, with the exception that the bottom of the wet pan is not perforated.

**279. Molding.**—Building brick are molded by the soft-mud, the stiff-mud, or the dry-press process. The stiff-mud process is employed in making nearly all other structural clay products. Fire-brick is sometimes made by the soft-mud process and roofing, floor, and wall tile by the dry-press process.

In the soft-mud process the clay, or mixture of clayey materials, is tempered with enough water to form a mass of soft consistency. The mixture is then introduced into wooden molds which are lined with sand to avoid sticking. Either hand or machine molding is employed in this process. Soft-mud brick exhibit five sides to which more or less sand adheres. The soft-mud process can be used with a greater variety of clays



FIG. 3.—A Small Auger Equipped with Device for Making Side-cut Brick.

than any other method of molding and very uniform brick may be made with it.

Less water is used in the stiff-mud process so that the mixture is much more rigid than in the soft-mud process. It is well applied to mixtures which are of medium plasticity. The clay from the pug-mill is forced through a tapered die by means of an auger and the issuing bar is cut into the required lengths. Owing to the motion imparted by the auger and friction on the sides of the die, various portions of the clay bar are given different velocities; consequently more or less laminations are present in the product of auger machines. Brick are made either end or side cut by this machine. Fig. 3 shows a side-cut machine. Auger brick machines have considerably greater capacities than the machines of the vertical press type used in the soft-mud or dry-press processes. For molding common brick, machines equipped with double or triple dies are sometimes used. Partition tile, conduits, fireproofing and the smaller sizes of drain tile are molded in a similar manner to stiff-mud brick. For



sewer pipe and the larger sizes of drain tile, vertical presses are employed. The uniformity in shape and size of stiff-mud or soft-mud brick can be greatly increased by repressing. Repressing increases the density, makes the surfaces smoother and harder, and in some instances, has improved the strength of brick. This auxiliary process is often employed in making high-grade face brick and in molding paving brick.

In the dry-press process the proportion of water is small enough to permit pulverizing the clay. The powdered mix is screened through a No. 16 sieve and then fed into molds on a vertical press. The molds are made of hard steel and steam-heated to prevent adherence of the clay. Vents are provided to allow the escape of entrapped air. Owing to the heavy pressures used, it is possible to obtain pieces with sharp corners and of much more uniform shape than can be gotten from auger machines. This process also does away with air drying although considerable free water must be driven off in the kiln. Face brick are commonly made by this process.

Terra-cotta is generally hand-molded or is cast into molds made of plaster of Paris.

**280. Drying.**—Great care is required to dry soft mud and stiff-mud products at maximum rate without causing checking. In the old method of drying, still employed at many brick yards, the ware is dried by the sun. Many plants provide permanent sheds with roofs which can be opened or closed. At some yards open-air driers are provided with artificial means for heating during bad weather.

Artificial driers are of two types, the hot-floor drier and the tunnel drier. The former is the older and is used for fire-brick, clay pipe and terra-cotta. The hot-floor drier is heated either by a furnace placed at one end of the drier or by exhaust steam from the engine used to furnish power. Tunnel driers are periodic—filled, dried, and emptied in rotation—or continuous—the green ware being loaded into one end of the tunnel and the dried product removed at the other. Tunnel driers are heated by flues underneath, by steam pipes, or by hot air from cooling kilns. They are more economical than hot-floor driers. In artificial driers the temperature rarely exceeds 120° C. The time required in drying varies from one to three days, depending upon the temperature of the drier, the character of the clay, and the shape of the body.

In some brick plants the green ware is set in a kiln and dried by wa heat from cooling kilns. This method requires more kilns, but effects saving in handling of brick.

**281. Kilns.**—In the brick industry four types of kiln are in use: scove kiln, the up-draft kiln, the down-draft kiln, and the continuous ki Of these types the down-draft and continuous kiln are used in burni other clay products. Terra-cotta is generally burned in round dc



FIG. 4a.—Setting a Scove Kiln. Temporary End and Side Walls Not in Place. (*Bull. No. 15, Wis. Geol. and Nat. Hist. Surv., Pl. 6.*)



FIG. 4b.—Circular Down-draft Kiln Used for Burning Brick. (*Bull. No. 15, Wis. Geol. and Nat. Hist. Surv., Pl. 6.*)



FIG. 4c.—Hugh Continuous Kiln (American Clay Machinery Co.)

FIG. 4.—COMMON TYPES OF KILNS.

draft kilns provided with muffles to prevent contact of the flame with the ware.

The scove kiln shown in Fig. 4a is much used in burning common brick. Brick are laid about 40 courses high and the entire kiln enclosed with a course or two of special brick containing a small proportion of coal. The outside is then plastered with mortar and fires started in the arches. Wood or oil is used as fuel. The temperature of the outer and inner portions of the kiln is equalized to some extent by the combustion of the coal-brick. Although the cost of the scove kiln is low, the loss in brick and in heat is high.

Up-draft kilns provided with permanent walls and roof are an improvement over the scove kiln, but they are more wasteful of heat and are heated less uniformly than the down-draft kiln.

The down-draft kiln (Fig. 4b) is either rectangular or circular in plan, the former being used largely in burning the better grades of building brick and in burning paving brick. As the name implies, the heat from fire after passing through up-takes to the top of the furnace is drawn downward through the kiln and passes through flues in the floor. Thence the waste gases are led through tunnels either to a stack passing up through the center of the kiln, or to a detached stack serving several kilns. With the down-draft kiln a more uniform distribution and better regulation of heat can be obtained than with either of the previously described kilns. Since the hardest burned ware, which becomes the softest in burning, is found at the top of the kiln, it cannot be misshapen by the weight of overlying courses.

Continuous kilns (Fig. 4c), are often built oval in plan and divided by vertical walls into a large number of compartments. These are loaded from the interior of the ring and unloaded from the outside. Each compartment is provided with an adjoined fire-box, a flue leading to a central stack and by-passes in the side walls through which the compartment can be connected with the adjacent compartments. Pockets are also provided in the top of each chamber for additional fires. Consider a chamber which has been loaded with ware having an unloaded compartment at the left. The chamber is first isolated from adjoining compartments and fired, until the most of the combined water is driven out of the ware, by heat from the side fire-box or wicket. After this stage has been passed, waste heat from the compartment on the right is admitted until a red heat is attained; then the top pockets are fired by means of slack coal. By this method, chamber No. 1 will be completely burned while in No. 8 or 9 setting will be in progress and the intermediate chambers are in various stages of burning. The greatest advantage of this type of kiln is the efficient utilization of fuel. It is used to burn brick, fire-proofing and tile.

**282. Burning.**—The burning of clays may be divided into three main stages: (1) dehydration or “water smoking,” (2) oxidation, (3) vitrification, or period of settlement in the kiln.

During the dehydration period the water which has been retained in the pores of the clay after drying is driven off, some of the carbonaceous matter is burned, a portion of the sulphur is distilled from pyrites, hydrous minerals like kaolinite and ferric hydroxide are dehydrated, and the carbonate minerals are more or less decarbonated.\* The speed with which these eliminations occur is dependent upon the water and mineral content of the clay, its porosity, and its texture, and upon the method of handling the kiln. Too rapid heating causes cracking or bursting of the ware. On the other hand, if alkali is contained in the clay or much sulphur is present in the coal, too slow heating produces a scum on the surface of the product. During the “water-smoking” stage, frequent measurements of the draft and temperature are made in order to standardize the procedure for a given clay. This period is generally completed before a temperature of 700° C. is reached.

During the oxidation period, which is nearly always completed at 900° C., the remainder of the carbon must be eliminated and, to promote stability, the ferrous iron must be oxidized to the ferric form. Although some of these changes begin before the completion of the preceding change, it has been pretty well demonstrated that the removal of sulphur cannot be completed before the carbon has been eliminated. Sulphur, on account of its affinity for oxygen, also holds back the oxidation of iron. Consequently, in order to avoid black or spongy cores, oxidation must proceed at such rate as will allow these changes to occur before the heat becomes sufficient to soften the clay and close its pores. Grog or sand is often added to the raw clay to produce a more open structure and thus provide for the escape of gases generated in burning.

The different stages in vitrification have already been mentioned. It should be borne in mind that but few clay products are vitrified to the point of viscosity, indeed, many common brick and tile cannot be called vitrified in any sense of the term. On the other hand paving brick must be burned to the stage which Wheeler calls complete vitrification if the maximum combination of hardness and toughness is to result. There is, consequently, a wide range in the maximum burning temperature to which the clay must be submitted depending upon the character of the raw material and the purpose of the ware. In the manufacture of tile and building brick this range varies between 900 and 1200° C.

The degree of burning is frequently determined by the settlement of the goods in the kiln, in some cases by pyrometers. For ware which is to be completely vitrified, it is advantageous to have the points of

\* Prof. E. Orton, Jr. *Trans. Am. Ceramic Soc.*, Vol. 5, p. 393.

incipient and viscous vitrification separated as widely as possible. This is desirable in order that goods from different parts of the kiln may not vary widely owing to non-uniform distribution of heat.

**283. Glazing.**—By glazing clay products, it is possible to give a pleasing appearance to the ware, to increase its imperviousness to water, or to accomplish both of these ends without incurring the cost of complete vitrification. Slip-clays, which have a high fluxing content and which may be so adjusted in composition that different coloring effects can be produced, lead compounds, barium compounds, and common salt are among the substances used for glazes. Decorative terra-cotta goods are sprayed with a thin mixture of slip-clay before burning. Sewer pipe is salt-glazed by the addition of common salt to the fires during the vitrification stage. Enamel brick are given a coating of slip containing oxide of tin or similar compound to render the glaze opaque.

**284. Flashing.**—Colors ranging from gold to dark reddish brown may be produced on many kinds of ware by flashing. The process consists in exposing the ware to a reducing atmosphere during a part or whole of the burning period. Front brick are often so treated with pleasing results. Flashing, however, is likely to deceive even experts concerning the degree of burning.

**285. Annealing.**—Great care is necessary in cooling the goods below a cherry-red heat in order to avoid checking and cracking. Hastening the annealing process may destroy the product of an otherwise successful burn. To make paving brick of maximum toughness requires an annealing period of seven to ten days.

**286. Sorting.**—In up-draft kilns the over-burned ware is found at the bottom near the top of the arches and the under-burned material is at the top. The converse is true for down-draft kilns. In either kiln, the best of the product is found in the intermediate courses. In the manufacture of paving brick, high-grade building brick, and drain tile, these different classes of goods are separated in unloading the kiln and the poorer grades of material sold for different purposes. Over-burned paving-brick and building brick are used in sewer construction. Soft-burned paving brick are used for exterior walls in building construction, while the soft-burned building brick serve as filling.

#### METHODS OF TESTING STRUCTURAL CLAY PRODUCTS

**287. Tests.**—Two classes of tests are used in judging of the quality of clay products: (A) tests which may be readily made on the job; (B) those which require laboratory equipment. The field tests are: (1) appearance, (2) hammer test, (3) hardness, (4) absorption, (5) specific gravity. In the laboratory, the following additional tests are sometimes made: (6) crush-

ing. (7) transverse bending, (8) rattle test \* (paving brick), (9) abrasion (paving brick), (10) resistance to alternate freezing and thawing. 1

**288. Appearance.**—Shape, color, kiln marks, checks, lamination and blisters all form more or less valuable indications of quality. The color of the outside of the goods is often misleading in regard to the degree of burning. This also applies to the color of the interior, unless one is familiar with the changes which the given clay undergoes in burning. The presence of lime pebbles over  $\frac{1}{8}$  in. in diameter is undesirable. Black or spongy cores show improper burning as previously mentioned. In brick, well-defined kiln-marks indicate that the brick have been hard burned, but do not serve to distinguish between hard burned and over burned brick. Checks and cracks may be due to improper drying or annealing. They decrease the strength and resistance to frost; if well marked they are sufficient cause for rejection. Checks produced in burning generally indicate brittleness. Pronounced laminations in the cross-section of the ware are objectionable since they weaken the structure and lessen resistance to freezing. Broken layers on the surface of some pipe or drain tile are due to air imprisoned in molding. They are not to be located on the inner surface. When present on the outer surface they should not exceed  $\frac{1}{4}$  of the thickness of the ware, and when on the inner surface they exceed  $\frac{1}{4}$  of the diameter of the pipe.

289. The Hammer Test. This consists of a series of tests designed to determine the relative strength of the muscles of the hand and wrist. A small block of wood is placed on a table and the subject is asked to strike it with the hammer. The force of the blow is measured by a scale.

[illegible][illegible]

in the absorption test and the rate of filling are properties deserving of careful consideration in determining resistance to freezing (see Art. 264).

Experiments by Douty and Gibson \* show that a rather high percentage of absorption is favorable to the development of a good bond between cement, mortar and brick.

No standardized method of making the absorption test has been specified for all kinds of clay products. Brick and partition tile are generally tested whole, but square fragments weighing a couple of pounds each are often selected to represent drain tile in absorption tests. Whatever method of testing is employed the specimens are first dried to constant weight at a temperature just above the boiling-point of water. Weighing should be done on scales sensitive to one-tenth of one per cent of the weight of the specimen. One of three methods of procedure may now be adopted. (1) The specimen is immersed in water at approximately 21° C. (70° F.) for forty-eight hours. It is then freed of surface water by wiping with a trowel and again weighted. The difference in weight between the soaked and the dry specimen divided by the dry specimen and multiplied by 100 gives the per cent absorption. This method is very commonly used. (2) The dried test-piece is immersed in water at 21° C. which is raised to the boiling-point in a half hour, is allowed to boil for five hours, is then cooled to 21° C., surface dried and weighed as before. (3) The dried specimen is subjected to a vacuum and water admitted without changing the pressure. After immersion for twenty-four hours it is surface dried and weighed. This method is cumbersome and seldom used. Of these methods the first is the most used and appears to be the most valuable for determining absorption under atmospheric conditions. It does not, however, furnish as satisfactory a determination of porosity as either of the other methods.

**292. Specific Gravity.**—Roughly, the higher the specific gravity, the greater is the strength of clay products. This rule is modified, of course, by conditions in burning and by the degree of burning. Tests by Purdy and Moore † show that the specific gravity of a given clay decreases as the vitrification period advances.

The specific gravity is generally found by the following formula:

$$\text{Specific gravity} = \frac{\text{Dry weight}}{\text{Dry weight} - \text{Weight in water when saturated}}$$

**293. Strength Tests.**—The methods of making the crushing and transverse tests of brick and building tile are the same as outlined in Art. 112 to 121. The crushing test is generally made on half brick bedded flatwise,

\* *Proc. A.S.T.M.*, Vol. 8, p. 529.

† *Trans. Am. Ceramics Soc.*, Vol. 9, p. 203.

but this method is not satisfactory both on account of the uncertainty in determining the final load and on account of the lack of opportunity for shear failure. These considerations have led some experimenters to bed the brick on the edges. The strength of brick on edge is generally 15 to 20 per cent less than the strength of specimens bedded flatwise.

The crushing test affords a means of comparing the quality of brick or tile but is of no value in determining the strength of a wall, since the latter depends primarily on the strength of the mortar. As a criterion of structural strength for brick, the transverse test is of more value than the crushing test, since transverse failure in a wall or pavement is likely to occur on account of improper bedment. Brick which are improperly annealed or checked in air-drying exhibit a lower transverse strength than properly

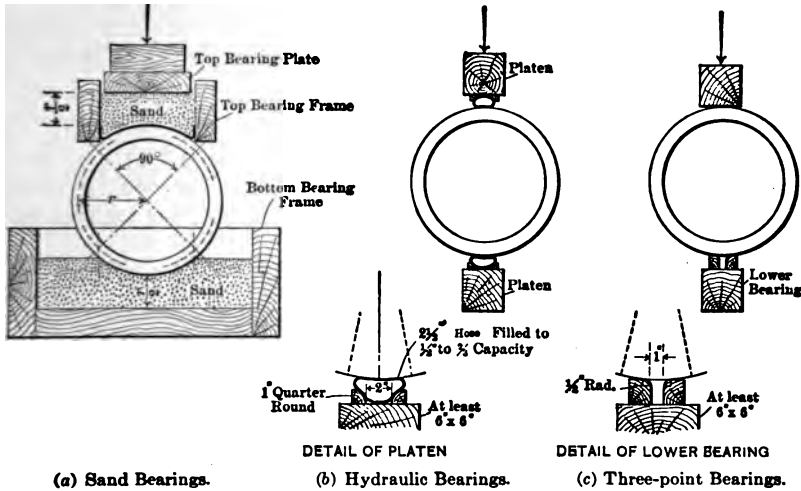


FIG. 5.—Methods of Supporting Drain Tile for Strength Tests.

treated specimens. Moreover, the transverse test can be made in a small machine without the expense of a bedment.

Drain tile and sewer pipe are often tested under gradually applied loads placed as indicated in Fig. 5. The tile should be thoroughly wet when tested and not less than five specimens should be tested. The supporting strength ( $W$ ) is defined as the load per lineal foot which the tile can carry when loaded through sand bearings. Under this condition the moment,

$$M = \frac{0.20RW}{12}, \text{ where } R \text{ is the radius of the center line of the tile in inches.}$$

To determine supporting strength from a three-point bearing test, multiply the breaking load per foot by 1.50; if hydraulic bearings are used multiply by 1.25. The three-point bearing method is simpler in application, but it is more severe on warped pipe than either hydraulic or sand bear-



ing. Sand bearings also approximate quite closely the loading conditions to which the pipe is subjected in a ditch.\*

**294. The Rattler Tests on Paving Brick.**—Although the rattler test has never been shown to be a true index of the life of brick in a pavement, yet as a means of standardizing resistance of paving brick to combined impact and abrasion it is considered valuable. The standard specification adopted by the American Society for Testing Materials is given in Appendix A.

**295. The Detroit Pavement Determinator.**—Recently considerable interest has been aroused in tests on sample pavements by means of a pavement determinator. The apparatus consists of a central vertical shaft which serves as an axis of rotation for a horizontal shaft. At each end of the horizontal shaft is mounted a heavy broad-faced wheel weighing 1650 lb. Each wheel is shod with five plungers which terminate in calks similar to those on a horse's shoe. The plungers are actuated by a cam gear and spring mechanism so that a heavy blow can be delivered to the pavement through the calks. The horizontal shaft is so driven that the wheels describe spiral paths over a ring 10 ft. 10 in. outside diameter by 8 ft. 7 in. inside diameter. The area is completely traversed in 330 revolutions. Wear is determined by measuring the depth to which the surface of the pavement is abraded.

Although a number of tests with this determinator have been run by the engineering department of the City of Detroit on different classes of pavements,† the results thus far published have not been sufficient to establish the value of the test. Tests of this character which will determine in a short space of time the wearing resistance of a pavement are desirable. On the other hand, owing to personal equation involved in laying the pavement, cost of the apparatus, and difficulties of standardization, it is doubtful if such a test will be widely used.

**296. Alternate Freezing and Thawing Test.**—Committee C-6 of the A.S.T.M. has prescribed an alternate freezing and thawing test‡ for drain tile which may be applied to other clay products. The test is made on three or more specimens from each of five separate tiles. The specimens are so chosen that they represent the ends and center of each tile. They must be sound, approximately square, and between 12 and 20 sq. in. in area. After being dried to constant weight at 110° C., or above, the specimens are cooled to 20° or 25° C., reweighed and immersed in pure water. The water is raised to the boiling-point and boiled for five hours, after which it is cooled to 10° or 15° C. The specimens are again weighed and

\* For more complete information see Standard Specifications for Drain Tile, Serial Designation: C4-16, A.S.T.M. Standards 1916.

† *Engr. Record*, Vol. 68, p. 457.

‡ See Std. Spec. for Drain Tile; Serial Designation C4-16.

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prise the poorer grades of building brick which are used for filling, backing, and in walls where appearance is of small moment. They often vary greatly in color, degree of burning and in shape. Front or face brick are made more carefully than common brick. They are generally pressed or repressed and are used in fronts of buildings and in walls for which a pleasing appearance is desired. Red, white, cream, buff-burning brick, and buff-burning brick speckled by the addition of manganese, all of which burn to an even tone, are much used as front brick. Tapestry brick, rug brick and stipple-faced brick are stiff-mud products which have had their edges roughened in different ways in order to create pleasing effects in wall constructions. Ornamental brick include enameled and glazed brick. The former constitutes the major portion of the ornamental brick produced in the United States.

**299. Requirements of Good Brick.**—The essential requirements for building brick are sufficient strength to carry the loads imposed in a structure, durability, and a pleasing appearance when exposed to view. Common brick of good quality should be free from checks or cracks, should emit a metallic ring when struck with the hammer and should exhibit a fine-grained, uniform, dense structure free from laminations or lime pebbles. Well-burned face brick should not be easily scratched by the knife. They should possess the previously mentioned characteristics, and also be uniform both in color and size. Regularity in size is of especial importance for brick which are to be used in highly stressed walls, since tests have shown that the strength of such masonry is reduced if the brick vary in size. Good brick should be free from soluble salts such as the sulphates of lime, magnesia and the alkalies, since these compounds are likely to produce efflorescence.

When uniformly loaded in compression an individual brick is, in general, sufficiently strong to withstand its proportionate part of any wall or pier load. However, owing to irregularities in the distribution of stress through the wall, the supporting capacity of a single brick may be as small as the transverse center load which it can carry when supported at the ends. Consequently it has been customary, whenever a strength test is specified for individual brick, to demand both crushing and transverse tests.

In a structure the durability of brick may be tested by frost action, by alternate wetting and drying, and by fire. Tests of fire resistance are generally made on wall-panels. Resistance to frost action is best ascertained from alternate freezing and thawing tests, which are, however, quite expensive. The absorption test has long been considered a measure of durability, although the basis for this assumption is questionable. In this connection it should be noted that resistance to freezing is dependent not only upon the porosity of the brick but also upon the size of the pores,

the size of the connecting canals, the strength, and the elasticity of the brick. If the pore space is not entirely filled with water, the common condition, the imprisoned air acts as a cushion and lessens the expansive effect of the freezing water. Thus far, no standards of strength or durability have been adopted for brick.

**300. Tests on Building Brick.**—In 1913 Committee C-3 of the American Society for Testing Materials proposed the following classification for building brick:

	Average Compressive Strength (Flatwise) on Five Samples, lb. per sq. in.	Average Absorption on Five Samples, per cent
Class A (Vitrified).....	5000 and over	< 5
Class B (Hard burned) .....	3500 to 4999	< 12
Class C (Common firsts) .....	2000 to 3499	< 18
Class D (Common) .....	1500 to 1999	

Through the aid of a number of laboratories the committee carried out an extensive series of tests part of which are reported in Table 1. Each result is the average of five tests. The compression tests were made on half brick bedded flatwise in plaster of Paris. Transverse specimens were placed flatwise on rocker bearings 6 in. apart and loaded at the center.

The modulus of rupture ( $S$ ) was calculated from the formula  $S = \frac{3 Pl}{2 bd^2}$ , where  $P$ =center load,  $l$ =span,  $b$ =breadth, and  $d$ =depth. These tests are very valuable since they afford a notion of the qualities of brick procurable in different parts of the country as well as the range in mechanical properties. From a consideration of the results in the table it appears that minimum moduli of rupture corresponding to classes A, B, C and D, 300, 400, 600 and 900 lb. per square inch, respectively.

Results of tests made at the Watertown arsenal indicate that the shearing strength of brick varies from 7 to 30 per cent of the compressive strength. At the University of Wisconsin paving brick tested in single shear with an apparatus similar to the Johnson shear tool (Art. 56) gave an average shearing strength equal to 17 per cent of the average compressive strength.\* The range in results and the low values obtained in the latter tests indicate that bending must have exerted a considerable influence in both of the above series of tests.

\* Thesis by W. T. Bolton and W. A. Goss, 1915.

(Each Value is the Average of 5 Determinations.)

**d** Tested endwise, 4280 lb. per sq. in.

TABLE 1.—(Continued.)

Name of Laboratory and Person in Charge of Tests.	Place of Manufacture.	Compressive Strength, Lb. per Sq. In.	Modulus of Rupture, Lb. per Sq. In.	Absorption, per cent.	REMARKS CONCERNING BRICK.	
					Kind.	Class.
Iowa State College: R. W. Crum	Mason City, Iowa.....	6,264	1440	10.28	Clay bk. stiff mud, side cut	A
	" " " " " "	5,984	1325	12.70	" " " "	A
	" " " " " "	4,564	1298	17.40	" " " "	B
	" " " " " "	5,094	1028	17.34	" " " "	A
	" " " " " "	" " " "	" " " "	" " " "	" " " "	"
Tulane University: W. B. Gregory and W. B. Koch	Plaquemine, La.....	3,293	517	21.54	Stiff mud, clay brick...	C
	New Orleans, La.....	3,881	256	21.20	" " " "	B
	Covington, La.....	3,567	255	27.50	Soft " " " "	B
	Pearl River, La.....	4,149	316	24.00	" " " "	B
	Fernwood, Miss.....	5,404	980	22.10	" " " "	A
	Slidell, La.....	4,454	577	20.20	Stiff " " " "	B
	Norfield, La.....	3,928	285	22.10	Soft " " " "	B
	Tangipahoa, La.....	4,341	667	17.80	" " " "	B
	Baton Rouge, La.....	2,364	323	26.80	" " " "	C
	Sibleyville, Ala.....	7,081	2066	6.67	Dry press clay brick hard	A
	" " " " " "	3,399	1229	10.18	" " " " soft	C
	Brookhaven, Miss.....	5,468	1190	17.35	Dry pressed.....	A
	Slidell, La.....	3,711	310	20.19	Stiff mud.....	B
	Novick, Ala.....	10,981	3354	4.35	" " " "	A
	Sibleyville, Ala.....	6,602	1160	10.83	Dry pressed.....	A
Massachusetts Institute of Technology: C. M. Spofford	" " " " " "	6,873	1183	8.31	" " " "	A
	" " " " " "	6,720	2119	7.50	" " " "	A
	W. Barnstable, Mass.....	1,564	281	17.90	Clay brick, soft mud...	D
	" " " " " "	3,718	452	15.00	" " " "	B
	" " " " " "	3,580	659	14.88	" " " "	B
	" " " " " "	5,302	851	10.66	" " " "	A
	Ganic, N. H.....	3,147	773	15.76	" " " "	C
	" " " " " "	3,092	812	15.50	" " " "	C
	" " " " " "	4,106	627	13.30	" " " "	B
	" " " " " "	4,474	845	12.21	" " stiff mud...	B
	" " " " " "	5,458	1342	10.80	" " soft mud...	A
	" " " " " "	4,780	937	8.36	" " " "	B
	" " " " " "	5,448	1420	6.93	" " stiff mud...	A
	" " " " " "	6,024	1522	4.58	" " " "	A
	Worcester, Mass.....	2,950	416	14.60	Sand-lime brick.....	C
University of Michigan: F. N. Meneff	Detroit, Mich.....	2,278	" " " "	" " " "	Clay brick, stiff mud...	C
	" " " " " "	2,398	" " " "	" " " "	" " soft mud...	C
	" " " " " "	1,846	" " " "	" " " "	" " " "	D
	" " " " " "	2,145	" " " "	" " " "	" " " "	C
	" " " " " "	" " " "	" " " "	" " " "	" " " "	"
University of Pittsburgh: J. H. Smith	Watertown, Pa.....	4,542	438	13.90	Sand-lime brick.....	B
	" " " " " "	4,334	592	12.30	" " " "	B
	Kittanning, Pa.....	12,780	2329	2.06	Clay brick, stiff mud...	A
	Darlington, Pa.....	9,266	2166	6.04	" " " "	A
	Beaver Falls, Pa.....	7,556	1438	6.12	" " " "	A
	Bradford, Pa.....	6,122	1124	8.95	" " " "	A
	" " " " " "	7,742	2058	6.84	" " " "	A
University of Pennsylvania: H. C. Berry	Pittsburgh, Pa.....	9,288	1275	10.17	" " " "	A
	Scranton, Pa.....	4,758	1609	8.26	Clay brick.....	B
	" " " " " "	5,718	1769	7.15	" " " "	A
	" " " " " "	9,728	2705	5.37	" " " "	A
	" " " " " "	9,052	2226	4.61	" " " "	A
University of Washington: A. H. Fuller	Spokane, Wash.....	4,156	564	17.90	Clay brick, salmons....	B
	" " " " " "	6,250	723	14.53	" " medium reds....	A
	" " " " " "	7,458	918	12.84	" " dark reds....	A
	" " " " " "	7,480	1105	9.01	" " hard burned....	A

Fig. 6 shows stress-deformation curves for three grades of brick tested on end. The softer varieties of brick exhibit stress deformation curves of sharper curvature. The modulus of elasticity of such brick generally lies between 1,500,000 and 2,000,000 lb. per square inch.

**301. Specific Gravity of Brick.**—The specific gravity of brick ranges from 1.9 to 2.6 depending upon the character of the raw materials and the

degree of burning. Brick made from impure fire-clays generally have a lower specific gravity than those made from shales. Some tests appear to indicate that the specific gravity decreases as the vitrification period progresses.

**302. Crushing Tests on Brick Piers.**—The main conditions governing the strength of brick piers under concentric loading are: (1) the strength of brick, (2) the strength and elasticity of mortar, (3) quality of workmanship in laying, (4) method of laying, (5) regularity in form of brick. The stiffness of piers is dependent largely upon the modulus of elasticity of

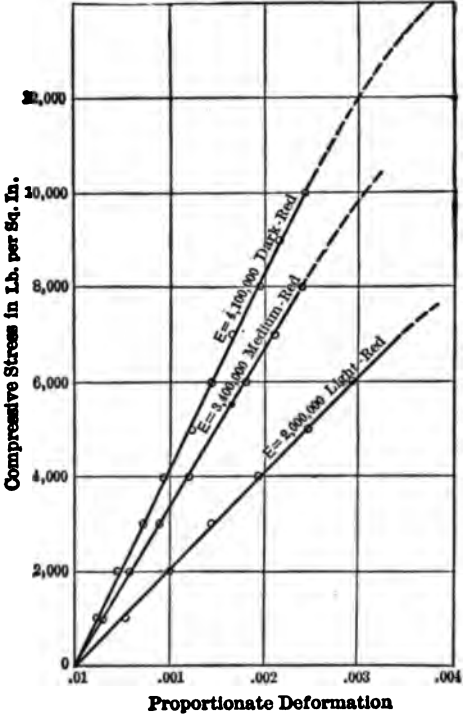


FIG. 6.

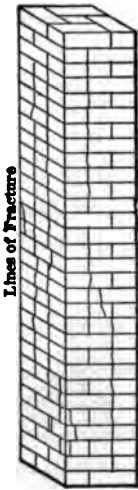


FIG. 7.

FIG. 6.—Elastic Properties of Common Brick Used in Pier Tests. The average crushing strength of these three grades of brick, crushed endwise, was 14,000, 10,500, and 7500 lb. per square inch, respectively. (*Tests of Metals*, 1885, p. 1138.)

FIG. 7.—Showing Method of Failure of Brick Piers. (*Tests of Metals*, 1883.)

the mortar, the quality of workmanship in laying, and upon the modulus of elasticity of the brick.

Tests at the Bureau of Standards Laboratory, at Pittsburgh, on large piers made of Class A (vitrified), Class B (hard burned), and Class C (common firsts)\* show very clearly the superiority of the piers made of

\* See classification in Art. 300.

the stronger brick (see Table 2 \*). The same result is also apparent in the tests by Talbot † summarized in Table 3. Fig. 7 shows the typical method of failure of brick columns. Transverse failure of the individual brick commonly begins when 40 to 60 per cent of the maximum load has been applied. The pier then gradually separates into a number of slender columns and failure finally ensues.

TABLE 2.—THE COMPRESSIVE STRENGTH OF LARGE BRICK PIERS

[Tests made at the Pittsburg Laboratory of U. S. Bureau of Standards.]

Piers were approximately 30×30 in.×10 ft. with joints  $\frac{1}{8}$  in. thick.

Lab. No.	Classification of Brick in Column According to		Characteristics of Mortar.		Age in Mo.	Arrangement of Courses of Brick. a : b : c.*	Ultimate Strength in Lb./In. <sup>2</sup>
	A.S.T.M. Committee of 1909	Trade Name.	Kind of Cement.	Mix Ct. : Sd.			
B 6	C	Common	Lime	1 : 6	4	1 : 1 : 5	126
B 5	C	Common	Lime	1 : 6	4	1 : 1 : 2	178
B 4	C	Common	Lime	1 : 6	4	1 : 0 : 1	210
B 12	B	Hard	Lime	1 : 6	4	1 : 1 : 5	990
B 11	B	Hard	Lime	1 : 6	4	1 : 1 : 2	890
B 10	B	Hard	Lime	1 : 6	4	1 : 0 : 1	840
B 9	A	Vitrified	Lime	1 : 6	4	1 : 1 : 5	1360
B 8	A	Vitrified	Lime	1 : 6	4	1 : 1 : 2	1270
B 7	A	Vitrified	Lime	1 : 6	4	1 : 0 : 1	1450
B 22	B	Hard	<div> <div>15% {</div> <div>Lime</div> <div>+85% {</div> <div>Port-</div> <div>land.</div> </div>	1 : 3	1	1 : 1 : 5	1760
B 28	B	Hard		1 : 3	1	1 : 1 : 2	870
B 20	B	Hard		1 : 3	1	1 : 0 : 1	1760
B 16	A	Vitrified		1 : 3	1	1 : 0 : 1	2900
B 3	C	Common	Portland	1 : 3	1	1 : 1 : 5	650
B 2	C	Common	Portland	1 : 3	1	1 : 1 : 2	560
B 1	C	Common	Portland	1 : 3	1	1 : 0 : 1	510
B 27	B	Hard	Portland	1 : 3	1	1 : 1 : 5	870
B 18	B	Hard	Portland	1 : 3	1	1 : 1 : 2	2070
B 17	B	Hard	Portland	1 : 3	1	1 : 0 : 1	2000
B 15	A	Vitrified	Portland	1 : 3	1	1 : 1 : 5	2900
B 14	A	Vitrified	Portland	1 : 3	1	1 : 1 : 2	2740
B 13	A	Vitrified	Portland	1 : 3	1	1 : 0 : 1	2710

\* a = header course, b = backup course, c = stretcher course. 1 : 1 : 5 indicates 1 header, 1 backup and 5 stretcher courses in sequence, etc.

\* *Engr. Record*, Vol. 71, p. 460.

† *Bulletin* No. 27 of the University of Illinois.



TABLE 3.—AVERAGE RESULTS OF COMPRESSIVE TESTS ON BRICK COLUMNS

[See *Bulletin* No. 27, Engr. Expt. Sta., Univ. of Ill.]Columns were  $12\frac{1}{2} \times 12\frac{1}{2}$  in.  $\times$  10 ft. and consisted of 40 to 43 courses of brick laid with  $\frac{1}{2}$ -in. joints.

No. of Tests	Characteristics of Columns.			Average Ultimate Strength Lb./In. <sup>†</sup>	Proportion of Load at which Popping Occurred ‡	Ratio of Strength of Columns to Strength of			Modulus of Elasticity in 1000 Lb./In. <sup>§</sup>
	How Laid.	Mix of Mortar.*	Age Days.			Brick	First Set.	6-in. Mortar Cubes.	
SHALE BUILDING BRICK									
3	Well	1 P : 3S	67	3365	.56	.31	1.00	1.17	4783
2	Well	1 P : 3S	181	3950	.....	.37	1.18	.....	5025
2	Well	1 P : 3S	68	2800 <sup>1</sup>	.66	.26	.83	.....	4400
2	Poorly	1 P : 3S	67	2920	.62	.27	.87	1.05	3525
2	Well	1 P : 5S	65	2225	.52	.21	.66	1.30	3250
1	Well	1 N : 3S	67	1750	.40	.16	.52	5.75	800
2	Well	1 L : 2S	66	1450	.43	.14	.43	....	104
UNDERBURNED CLAY BRICK									
2	Well	1 P : 3S	63	1060	.76	.27	.31	.37	433

\* P=Portland cement; S=sand; N=natural cement, L=lime.

† The maximum range in strength for any set of columns was less than 13 per cent.

‡ Popping was probably due to transverse rupture of brick.

<sup>1</sup> These columns were loaded with 1-inch eccentricity.

Owing to differences in ratio of height to least width, one would expect a brick tested flatwise to have about 40 per cent more strength than a solid homogeneous pier of the same material having a ratio of height to least breadth of from 6 : 1 to 10 : 1. Therefore a pier which developed 70 per cent of the crushing strength of the component brick would be ideal. Since, however, the mortar of the joints never possesses the same strength and elastic properties as the brick composing the pier, this condition is never realized. Yet on account of the thinness of the joints it is reasonable to expect the strength of the joint to exceed the strength of cubes made from the mortar. The results in Table 2 show that piers of Class A or Class B brick laid in 1 : 3 Portland cement mortars were twice as strong as similar piers laid in lime mortar. The low strength of the lime mortar piers is partly attributed to the fact that the lime mortar in the central portions of the joints did not receive sufficient air to become carbonated. Talbot's tests indicate that the columns laid in 1 : 3 Portland cement mortar were 51 per cent. stronger than those laid in like mortar of 1 : 5 proportions, 92 per cent. stronger than those laid in 1 : 3 natural cement

mortar and 132 per cent stronger than those laid in 1 : 2 lime mortar. Tests made at the Watertown Arsenal \* clearly show the advantage of using mortar stronger than a 1 : 3 mix with very hard burned brick. In tests of piers of wire-cut brick having a crushing strength of approximately 13,000 lb. per square inch those laid in neat Portland cement had a strength of 31 per cent of the brick, whereas those laid in 1 : 3 Portland cement mortar only possessed 19 per cent of the strength of the brick.

The advantage in stiffness arising from the use of strong mortar in the joints is also well illustrated in the tests by Talbot.

Tests at the Watertown Arsenal indicate that the strength of small columns (12×12 in.) is 40 to 50 per cent greater when the brick are laid on edge than when laid flatwise. This shows the desirability of increasing the thickness of brick which are to be used in heavily loaded piers.

Some tests at Cornell University † and others made at the Bureau of Standards ‡ demonstrate that the strength of a column may be increased by placing a horizontal layer of wire meshing in every joint. A decrease in strength obtained, however, when the four or more courses intervened between layers of meshing.

**303. Resistance of Brick Walls to Fire.**—In a series of fire tests on 6×9-ft. wall panels at the Underwriters' Laboratories, in Chicago, brick panels showed marked resistance to fire and low conductivity. Tests were made on a 12-in. wall of well-burned Chicago brick, an 8-in. panel of hydraulic-pressed brick from Indiana, an 8-in. panel of common brick from St. Louis, and an 8-in. panel of sand-lime brick from Indiana. The strengths and absorptive properties of these bricks are given in Table 4. With the exception of the panel made of hydraulic brick which was laid in lime putty, cement mortar was used. The tests were made by subjecting the face of the panel to a temperature which rose to about 800° C. in one-half hour and then varied between 800 and 900° C. for one and one-half hours at which time the panel was removed and the face quenched by water from a fire-hose. Each high temperature recorded in Table 4 represents an average of a number of pyrometer readings, the thermo-couples being arranged to secure the variation in temperature at different parts of the furnace and at various points on the face of the wall.

The panels of clay brick withstood the tests better than the panel of sand-lime brick. The panel of hydraulic-pressed brick was in the most perfect condition after quenching. About 18 per cent of the brick on the face of this panel were cracked through and a very few were spalled. The Chicago brick contained lime knots which caused a large percentage of the exposed brick to crack when quenched. In 60 to 70 per cent of these the

\* Tests of Metals, 1904.

† *Trans. Asso. Civ. Engrs.*, Cornell University, Vol. 8, 1900.

‡ *Engr. Record*, Vol. 71, p. 460.

TABLE 4.—EFFECTS OF HIGH TEMPERATURE ON BRICK AND BRICK WALLS

[Bulletin No. 370 U. S. Geological Survey.]

Wall panels fired for two hours, for one hour at maximum temperature.

Kind of Brick.	Common Clay.	Hard-burned Clay.	Hyd-pressed Brick.	Sand-Lime.
Thickness of panel, in. ....	8	12	8	8
Maximum average temperature on exposed face in ° C. ....	850	770	850	790
Maximum average temperature on unexposed face after 2 hr. in ° C. ....	58	18	35	55
Mean temperature of air on unexposed face. ....	23	3	3	19
No. unexposed brick tested transversely. ....	5	2	2	5
Average modulus of rupture of unexposed brick, lb./in. <sup>2</sup> ....	1178	482	718	319
No. exposed brick tested transversely. ....	0	0	0	5
Average modulus of rupture of exposed brick, lb./in. <sup>2</sup> ....	.....	.....	.....	56
No. unexposed brick crushed. ....	5	13	13	5
Average crushing strength of unexposed brick, lb./in. <sup>2</sup> ....	3866	2729	4440	2035
No. exposed brick crushed. ....	0	13	13	5
Average crushing strength of exposed brick, lb./in. <sup>2</sup> ....	.....	2793	3701	1750
No. unexposed * brick immersed. ....	5	2	2	5
Average per cent absorption, unexposed brick, after forty-eight hours' immersion. ....	6.2	20.2	10.7	15.9

\* Absorptions of exposed brick were about the same as for unexposed brick.

cracks were sufficiently large to permit picking off portions of the brick. About half of the faces of the St. Louis brick were so cracked at the conclusion of the test that they could be readily removed, and the face of the wall was discolored to a depth of about 1 in. After firing, the face of the panel of sand-lime brick looked soft and chalky. It was washed away to a depth of  $\frac{3}{8}$  to  $\frac{1}{2}$  in. when the hose was applied. Only about 20 per cent of the exposed brick could be removed from the wall intact.

#### SAND-LIME BRICK \*

Although not a clay product, the fact that a million dollars' worth of sand-lime brick are annually used as a substitute for clay brick warrants a brief discussion of their manufacture and properties in this chapter.

**304. Definition.**—Sand-lime bricks are made from a lean mixture

\* For further information on sand-lime brick see *Cements, Limes and Plasters*, by E. C. Eckel. An interesting article on *The Chemistry of Sand-lime Brick*, by T. R. Ernest appears in *Trans. Am. Ceramic Society*, Vol. 13, p. 649.

of slaked lime and fine silicious sand, molded under mechanical pressure and hardened under steam pressure.

**305. Manufacture.**—The sand should be free from clay or mica. It should all pass a 20-mesh and three-fourths of it should be retained on 60-mesh. If the grains run larger than a 20-mesh, the coarse particles must be screened out or reduced in a tube-mill. A high-calcium lime is referred to a brown or dolomitic lime owing to the rapidity with which the former hardens. The requisite percentage of lime varies between 4 and 10 per cent. Generally the lime is slaked before mixing with the sand by placing it beneath the brick cars in the hardening cylinder and allowing the steam to act upon it. Often the sand and lime are mixed dry in a tube-mill and the mixture is then tempered in a special type of pug-mill. In some plants the quick-lime is slaked, the sand ground, and an intimate mixture secured by running the wet sand and lime through a wet and dry grinding mill, an apparatus similar to a dry-pan. Molding is done in powerful presses, some of which can exert a compression of 20,000 lb. per square inch on the surface of the brick. After removal from the press the bricks are stacked on cars which are run into the hardening cylinders. In the latter a steam pressure of 100 to 150 lb. per square inch is used. After hardening for six to ten hours the bricks are ready for use.

**306. Comparison of Clay and Sand-lime Brick.**—On account of their smooth surfaces, even shape, freedom from efflorescence, uniform color and satisfactory strength, sand-lime bricks are sometimes preferred to clay bricks. Some of the American sand-lime bricks, however, have not been as durable when exposed to the weather as good grades of clay bricks. Unless made with great care sand-lime bricks do not resist frost action or fire as well as clay bricks.

**307. Physical Properties of Sand-lime Brick.**—Good sand-lime bricks pressed flatwise have a compressive strength between 2500 and 4000 lb. per square inch. In cross-bending the modulus of rupture should exceed 150 lb. per square inch. After immersion for forty-eight hours good bricks should not show more than 15 per cent absorption. The specific gravity generally lies between 2.1 and 2.3.

#### PAVING BRICK \*

**308. Manufacture.**—Paving bricks are made from three classes of clay, surface clays, impure fire-clays and shale. Owing to the narrowness of the vitrification range for most surface clays, they are apt to produce either underburned or overburned bricks. Impure fire-clays make a good

\* Additional information on methods of manufacture may be gotten from *Vitrified Paving Brick*, by H. A. Wheeler, Randall & Co.; and from *Burning Brick in Down-draft Kilns*, by W. D. Richardson, Randall & Co.

brick when sufficiently vitrified, but require a high temperature. The shales are by far the best source of raw material for paving brick. Wheeler suggests that shales suitable for paving brick should approximate the following analysis: Silica ( $\text{SiO}_2$ ) = 56, alumina ( $\text{Al}_2\text{O}_3$ ) = 22, ignition loss = 7, lime ( $\text{CaO}$ ) = 1, magnesia ( $\text{MgO}$ ) = 1, alkalis ( $\text{K}_2\text{O}$  and  $\text{Na}_2\text{O}$ ) = 4 per cent. Fluxing impurities in the above analysis total 13 per cent.

Three types of brick are molded with the stiff-mud process; wire-cut, wire-cut lug, and repressed brick. Wire-cut lug brick are side-cut by a vertical wire. This wire is guided by slots in horizontal plates located above and below the bar of clay. The arrangement of the slots is such that four lugs are formed on one side of each brick; the other side is cut plane. The brick are laid in the pavement so that the smooth side of one brick is in contact with the lugs on the adjacent brick. It is claimed that the vertical joints are much stronger in pavements made of this brick than in those made of plane-cut brick owing to the superior bond between the filler and the lug surfaces. Other types of stiff-mud paving brick may be either end or side-cut.

Paving brick are generally burned in down-draft or continuous kilns. From seven to ten days are required in burning and a like period for proper annealing. The temperature required to bring shales to complete vitrification (Wheeler), is 850 to 1100° C. Impure fire-clays require a temperature from 100 to 200° C. higher.

By employing impure fire-clay, with which danger of over-burning is small, as high as 90 per cent of first-class paving brick may be produced. Using shale it is not possible to average more than 75 per cent of first-class pavers.

**309. Requirements of Good Paving Brick.**—Every brick should be free from marked distortion, should have one plane edge, and should be free from cracks, checks, and blisters. It should give a high metallic ring when struck with a hammer and should, when broken, exhibit a uniform close-grained structure free from laminations. The interior of a properly burned brick cannot be scratched with a knife. *Uniformity in quality is of vital importance in order that the pavement may wear evenly.*

**310. Physical Properties of Paving Brick.**—Good brick should have a crushing strength in excess of 8000 lb. per square inch. Overburned brick are likely to have a very high crushing strength. The transverse strength (modulus of rupture) should not be less than 1500 lb. per square inch. Low transverse strength may be due to either overburning or improper annealing. Shale brick when properly vitrified generally absorb between 0.5 and 2 per cent; brick made of impure fire-clay may absorb as high as 5 per cent. An absorption less than 0.5 per cent generally denotes over-burning. In specific gravity, shale pavers commonly range

between 2.2 and 2.5, impure fire-clay between 2.1 and 2.3. The better grades of paving brick lose less than 20 per cent by weight, in the rattler test. Wide variation in the losses of individual brick indicate non-uniformity in methods of manufacture and are sufficient cause for rejection of brick even though the average loss is small.

### REFRACTORY BRICK \*

**11. Introduction.**—Certain classes of brick are much employed to line hearths and the various classes of furnaces used in metallurgical processes. Such brick must be able to withstand high temperatures without undue softening or change in volume, must resist the action of gases and slags generated during the process, must resist abrasion when hot, must possess low thermal conductivity. In accordance with the character of the chemical reaction which different refractory brick resist, they are divided into three classes: acid, basic and neutral.

**12. Acid Brick.**—The brick which are commonly used to resist the action of silicious and other acid slags are fire-brick and silica brick.

Fire-brick are made from fire-clays which are sometimes grogged with 10 to 20 per cent of sand to reduce shrinkage. The clay generally consists of  $\frac{1}{2}$  to  $\frac{2}{3}$  silica,  $\frac{1}{3}$  to  $\frac{2}{3}$  alumina and less than 10 per cent of fluxes. More refractory brick are made from clays having low flux contents and high alumina contents. A very high alumina content, however, gives the brick a basic reaction. The brick are molded by either the soft- or dry-mud processes and repressed after partial drying.

First class fire-brick should have a compressive strength of at least 10,000 lb. per square inch and should withstand a load of 50 lb. per square inch at a temperature of 1350° C. without deforming over 11 per cent. They should not soften at a temperature less than 1700° C.†

**13. Silica Brick.**—Quartzite, sandstone, or silica sand, which consists of 95 per cent or more pure silica, are the main constituents used in making silica brick. If the silicious rock contains small percentages of iron it is sometimes possible to mold the brick without artificial additions. English ganister brick is made from such material. Ordinarily, however, the ground silicious rock or the sand are not sufficiently plastic when mixed with water and are adulterated either with very small percentages of fire-clay or about 1½ per cent of high-calcium lime. Silica brick are fired at temperatures slightly higher than fire-brick.

On account of their brittleness and expansibility when heated, silica brick must be laid with wide joints. The compressive strength of silica

For additional information on refractory materials reference should be made to Ward's *Refractories and Furnaces*, from which considerable of this material has been taken.

See *Technologic Paper No. 7*, U. S. Bureau of Standards.

The advantages of hollow block and fireproofing are lightness, low permeability to water, low heat conductivity and a rough surface to which plaster may be directly applied, thus avoiding the necessity of furring and lathing. Fire tests have shown, however, that partition tile and hollow blocks are likely to split at the junctions of webs and faces, especially when the hose is turned upon hot partitions.\* It is, therefore, safe practice to insist on fire tests of hollow block panels when they are to be used for fire protection.

Hollow blocks of good quality should have a compressive strength of 1000 lb. per square inch of gross section when the load is imposed in accordance with the scheme of design. Many of the best grades of hollow blocks will develop 2000 to 4000 lb. per square inch of gross section depending upon the direction in which the load is applied. After forty-eight hours immersion, good blocks generally absorb less than 12 per cent of water.

A notion of the variability of clay building tile and of the necessity for testing them is afforded by a large series of tests made at several laboratories for Committee C-10 of the A.S.T.M. (See *Proceedings*, Vol. 17, Pt. 1, p. 334.) The manufacturers were requested to select specimens which fairly represented the quality of their product. From the test data, the compressive strengths ranged from 640 to 12,360 lb. per square inch of net section, corresponding approximately to 95 and 6000 lb. per square inch of gross section, respectively. The largest value was recorded for a medium-burned tile tested on end and the smallest for a soft-burned tile loaded flatwise. Absorption, by weight, after boiling five hours, ranged from 1.8 per cent for a hard-burned specimen to 20.3 per cent for a very soft tile.

**317. Tests of Hollow Block Columns.**—Prof. A. N. Talbot reports two series of tests on columns of hollow blocks laid in Portland cement mortar in *Bulletin* No. 27 of University of Illinois Engr. Expt. Sta. In the earlier series the end faces of the blocks, which bore the load, were very uneven. This defect caused a marked difference in the strengths of the two series of columns. Most of the blocks were 8 in. high, 4 in. wide and 8 in. long. The cells occupied about 15 per cent of the volume of a block. From compressive tests on 33 single blocks the range in strength based on net area, was 3350 to 9070 lb. per square inch; the average was 5451 lb. per square inch. Transverse tests on seven blocks, tested with cells vertical, gave a range in modulus of rupture of 870 to 1240 lb. per square inch with an average of 1022 lb. per square inch of net section.

The seven columns tested in 1907 were approximately 12 ft. high, the twelve columns of the 1908 series were 10 ft. high. Columns of the earlier series ranged in cross-section from  $8\frac{1}{2} \times 8\frac{1}{2}$  in. to  $17\frac{1}{2} \times 17\frac{1}{2}$  in., all of the columns of the later series were  $12\frac{1}{2} \times 12\frac{1}{2}$  in. in cross-section. The

\* *Bulletin* 370, U. S. Geol. Survey.

columns of the earlier series were laid in 1 : 2 mortar. Most of the columns of the series of 1908 were laid in 1 : 3 mortar. The strengths of the columns tested in 1907 varied from 2710 to 3440 lb. per square inch; those tested in 1908 had strengths ranging from 3040 to 4300 lb. per square inch, the latter value being estimated. By reference to Art. 302 it will be noted that the strengths of these hollow block columns compare very favorably with the strengths of first-class brick columns. The initial modulus of elasticity of the columns varied between 1,910,000 and 2,860,000 lb. per square inch.

**318. Roofing Tile.**—For making roofing tile, the clay must not only be plastic, but it must dry and burn without suffering distortion. Considerable care is also required in preparing the raw material. After being ground in a dry-pan and finely screened the clay is pugged in a wet-pan and made into balls which are stored in bins until wanted. The balls are then taken to an auger machine and molded into tile or, if interlocking tile are to be made, they are formed into clots. The clots are then formed into tile on a press. Common

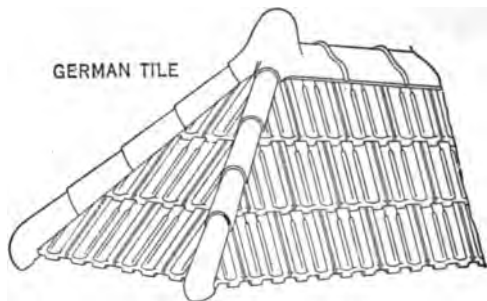
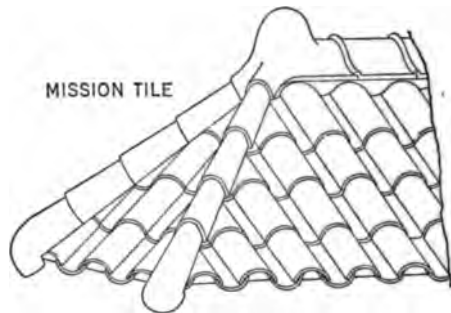
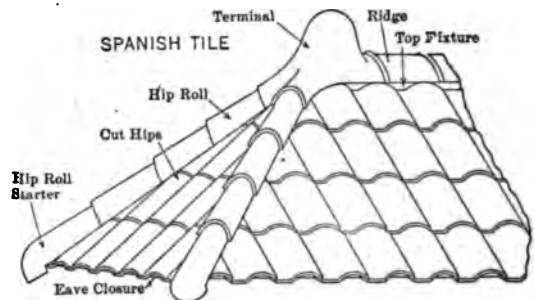


FIG. 9.—Examples of Various Roofing Tiles.



types of tile are shown in Fig. 9. After drying, roofing tile are burnt in saggars in down-draft kilns. Tile which are to serve as insulation against heat are soft-burned and porous. Where tile are to be subjected to freezing they are burned harder or glazed.

Roofing tile should be strong, durable, free from soluble salts, and impervious to water. Roofing tile, when properly made, form a strong, durable, fireproof roof which is a poor conductor of heat. The chief objections to tile roofing are the expense and the heaviness of the construction.

**319. Floor-tile.**—White-burning and red-burning clays, fire-clays and shales are used in making tile for floor surfaces. The chief considerations are color when burned, freedom from soluble salts, and absence of

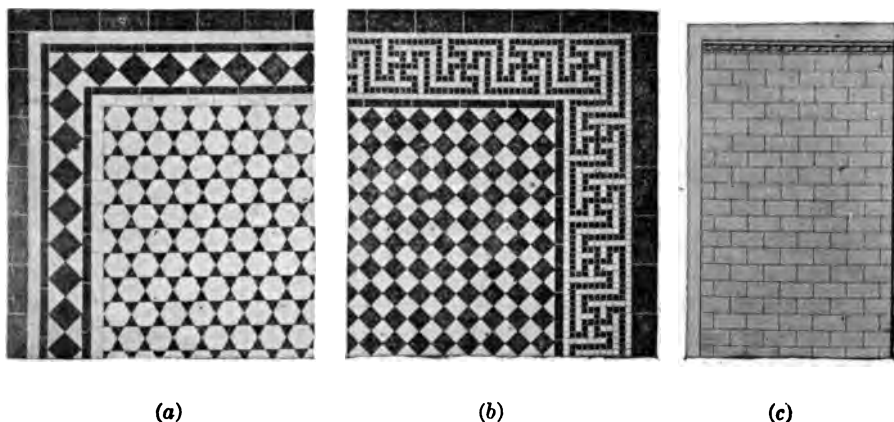


FIG. 10.—Floor Tile and Wall Tile Designs. (a) and (b) floor tile, (c) wall tile.

distortion and checking in burning. The dry-press process is used in molding, and down-draft kilns in burning.

Floor-tile may be divided into two classes in accordance with the method of molding the design into the tile. They are face-tile, commonly called encaustic tile because they have a burnt-in design, and plain tile.

Plain tile are composed of the same clay or mixture of clay throughout. Frequently these are made in the form of mosaics. They may be either vitreous or semi-vitreous.

Plain face-tile are made by covering the die forming the base of the mold with a thin layer of specially prepared clay, filling the mold with a clay backing and compacting in a hand press. In making inlaid face-tile, a cellular frame is placed on the bottom of the mold and clays which have been properly colored are screened into the different cells in accordance

with the pattern scheme. The frame is then removed, the backing inserted, and pressure applied to compact the tile.

Floor-tile should show little absorption, have a high transverse strength and a high resistance to abrasion.

**320. Wall-tile.**—Tile for surfaces of walls differ from floor-tile principally in design and degree of burning. Wall-tile are burned at a comparatively low temperature, glazed, and fired again in a muffle-kiln at a still lower temperature. Wide difference in color, in shades of a single color, and in relief design can be obtained. Wall-tile are much used in wainscotings and to some extent in arches and ceilings. Floor and wall-tile designs appear in Fig. 10.

### TERRA COTTA

**321. Decorative Terra-cotta.**—For the ornamentation of buildings and structures, decorative terra-cotta is now very widely used. It is made from a very finely ground mixture of fire-clay and shale, or fire-clay and impure clay, which is adulterated with ground brick or other burnt clay to decrease shrinkage. Plain forms of uniform design are made by hand in plaster molds, but more intricate designs are modeled in the green clay by expert workmen. Terra-cotta is dried very slowly to avoid warping and checking. It is then sprayed with a slip which lends a color to the goods on burning. For certain classes of goods glazes of a bright or dull finish are also applied. Burning is done in muffle-kilns at temperatures between 1100 and 1300° C. To obviate kiln-marking, each piece is so supported by fire-clay tile that it carries only its own weight. Fig. 11 shows certain types of terra-cotta forms.

**322. Terra-cotta Lumber.**—A very different material from decorative terra-cotta is the terra-cotta lumber used for floor arches and fireproofing (see top of Fig. 8). It is made in much the same way as partition tile with the exception that a considerable quantity of sawdust is incorporated in the raw mix. The goods are burned at a low temperature, which destroys the sawdust but leaves the product sufficiently soft and porous to permit cutting with a saw. The strength and lightness of this material are its chief assets. In compression, good material should withstand 2500 lb. per square inch of net section.

### CLAY PIPE

**323. Sewer Pipe.**—In the construction of sewers clay pipe has long been successfully used. Sewer pipe must have high strength in order that it may carry the ditch filling. To successfully withstand the action of acids and gases in the sewage, the pipe must be hard-burned and imper-

vious. A smooth interior surface which offers little resistance to flow is also desirable.

Sewer pipe is made from red-burning clays, fire-clays, shales and mixtures of shale and fire-clay. It is generally molded in vertical double-cylinder presses from a stiff-mud mix, although some pipe is made by the dry-press process. Sewer pipe is commonly burned in down-draft kilns at temperatures between 1050° and 1300° C. The term vitrified pipe is often a misnomer, since the burning temperature frequently is insufficient for vitrification. However, the firing should be carried to the stage of incipient vitrification. Salt glazing is generally practiced to insure a smooth and impervious surface. The pipe is made in lengths of about 3 ft. and in diameters up to 42 in. Sewer pipe are commonly provided with a bell on one end into which the small end of an adjacent pipe is fitted. Joints are filled with 1 : 1 Portland cement mortar.\*

In general, small fragments from good pipe will have an absorption less than 5 per cent after forty-eight hour immersion in water. Results of crushing tests in which the lower portion of the pipe was bedded in sand and the load applied on the top through a hardwood strip 1-in. wide appear in Table 5.

TABLE 5.—CRUSHING TESTS ON SEWER PIPE MADE BY THE MUNICIPAL SEWER-PIPE TESTING LABORATORY OF BROOKLYN, N. Y. IN 1909

[See *Municipal Journal*, Vol. 38, p. 160.]

Size of Pipe, In.	Total Number Tested.	Number Tested to Destruction.	Crushing Strengths in Pounds per Linear Foot of Pipe.			Required Strength, Lb. per Ft.†	Percentage of Failures.
			Maximum.	Minimum.	Average.		
6	170	169	2333	1033	1537	1000	0
12	245	245	2900	933	1542	1150	2.04
15	72	72	2800	1300	1935	1300	0
18	25	25	3100	1734	2389	1450	0
24	17	14	3800	2200	2825	2000	0
30	2	2	3280	3240	3260	.....	0
30*	2	2	3240	3080	3160	.....	0

\* Double strength pipe.

† Local standard.

**324. Drain Tile.**—Clay drain tile are made principally from shales and impure clays. The smaller sizes are molded in auger machines, but the larger ones are made in sewer-pipe presses. The tile are burned in a

\* The remarks in Art. 309 concerning appearance, hardness, texture, ring under the hammer of paving brick apply with equal force to sewer pipe and the better grades of drain tile.

variety of kilns sometimes with brick or fireproofing. Kiln temperatures are lower than those used in burning sewer pipe, although the better grades of tile are burned until steel-hard. Salt-glazing is sometimes employed. Drain tile are generally cylindrical in form and laid with uncemented butt joints through which the drainage water seeps.

The specifications which appear in the following table for the average strength of five specimens of drain tile were adopted by the A. S. T. M. in 1916 (see Standard Specifications for Drain Tile); the values for sewer pipe were proposed by Committee C-4 of the Society in the same year.

AVERAGE STRENGTH REQUIREMENTS FOR CLAY AND CEMENT DRAIN TILE AND SEWER PIPE

DRAIN TILE.				SEWER PIPE.	
Internal Diameter of Tile, In.	Minimum Average Ordinary Supporting Strength in Pounds per Linear Foot for			Internal Diameter of Pipe, In.	Pressure in Pounds per Linear Foot Applied through Knife-edge Bearings.
	Farm Drain Tile (a)	Standard Drain Tile (b).	Extra-Quality Drain Tile (c).		
4	800	1200	1600	6	900
6	800	1200	1600	8	900
8	800	1200	1600	10	1000
10	800	1200	1600	12	1050
12	800	1200	1600	15	1250
14	900	1200	1600	18	1400
16	1000	1300	1600	20	1550
18	.....	1400	1800	22	1750
20	.....	1500	2000	24	1950
22	.....	1600	2200	27	2150
24	.....	1700	2400	30	2350
26	.....	1800	2600	33	2500
28	.....	1900	2800	36	2800
30	.....	2000	3000	42	3200
32	.....	2100	3200		
34	.....	2200	3400		
36	.....	2300	3600		
38	.....	2400	3800		
40	.....	2500	4000		
42	.....	2600	4200		

(a) Farm drain tile for ordinary private drainage work on farms where the depth and size are moderate.

(b) Standard drain tile is for district drainage at moderate depths.

(c) Extra-quality drain tile is for district drainage where the depth is large and a first-class pipe is needed.

Three methods of testing tile are outlined in Art. 293.

**325. Conduit.**—In large cities considerable use is now made of clay conduits for carrying underground cables and wires. Hollow rectangular prisms rounded at the corners and traversed by several longitudinal ducts are much employed in lengths of about 3 ft. Conduits are made of the same material and molded in the same manner as fireproofing. They are, however, hard-burned and salt-glazed to render them impervious to water.

## CHAPTER IX

### PORTLAND CEMENT

**326. The Cements of Construction.**—The cementing materials which are most used in engineering constructions may be classified as follows:

1. Hydraulic cements	$\left\{ \begin{array}{l} \text{Portland cement} \\ \text{Natural cement} \\ \text{Puzzolan cement} \\ \text{Blended cement} \\ \text{Improved cement} \end{array} \right\}$	Will set under water
2. Limes	$\left\{ \begin{array}{l} \text{Quick lime} \\ \text{Hydrated lime} \\ \text{Hydraulic lime (sets under water)} \end{array} \right\}$	
3. Gypsum Plasters	$\left\{ \begin{array}{l} \text{Plaster of Paris} \\ \text{Cement plaster} \\ \text{Hard finish plaster} \end{array} \right\}$	
4. Bitumens		

Only the first three classes will be discussed herein; for the properties and uses of bitumens reference may be made to Blanchard and Drowne's *Textbook on Highway Engineering*, Baker's *Roads and Pavements*, or other standard works on pavements.

Owing to the widespread use and importance of Portland cement as a material of construction, and on account of the necessity for a thorough knowledge of its properties in order to properly fabricate it into structures, we shall consider at some length its nature, manufacture and properties.

#### NATURE OF PORTLAND CEMENT

**327. Definition.**—In the revised specifications for Portland cement\* the following definition appears.

“Portland cement is the product obtained by finely pulverizing clinker produced by calcining to incipient fusion, an intimate and properly proportioned mixture of argillaceous and calcareous materials, with no additions subsequent to calcination excepting water and calcined or uncalcined gypsum.”

\* See Art. 396.

**328. The Characteristics of Portland Cement.**—Although the characteristics of Portland cement are not measures of quality, they often serve to distinguish differences in brand and to separate Portland from other cements. Typical Portland cement is a flour-like powder which varies in color from a greenish gray to a brownish gray. At the present time, white Portland cement is also being successfully produced. In general, the specific gravity of Portland cement is higher than that of other hydraulic cements and lies between 3.10 and 3.15. Although its specific weight is quite variable, ranging from 75 to 95 lb. per cubic foot, depending upon the compactness, it is considerably higher than the specific weight of natural cement. Portland cement when mixed with water sets much more slowly than natural cement and more rapidly than puzzolan cement. It hardens more rapidly and acquires greater strength than any of the other hydraulic cements.

**329. The Chemical Elements in Portland Cement.**—The three fundamental constituents of hydraulic cements are lime, silica, and alumina. In addition to these most cements contain small proportions of iron oxide, magnesia, sulphur trioxide, alkalies and carbon dioxide. From the published analysis of a large number of cements, the table below, showing the range in composition and a typical analysis for American Portland cement, has been compiled.

Element	Range	Per cent	
		Average	
Lime (CaO).....	59-65	62-63	
Silica (SiO <sub>2</sub> ).....	19-25	22-23	
Alumina (Al <sub>2</sub> O <sub>3</sub> ).....	5-9	7-8	
Iron Oxide (Fe <sub>2</sub> O <sub>3</sub> ).....	1-5	2-3	
Magnesia (MgO).....	1-4	1.5-2.5	
Sulphur Trioxide (So <sub>3</sub> ).....	1-2	1.5	
Alkalies (K <sub>2</sub> O and Na <sub>2</sub> O).....	0-1	trace	
Water (H <sub>2</sub> O) and Carbonic Oxide (CO <sub>2</sub> ) .....	1-4		

Although it is certain that the way in which the chemical elements are combined in the cement exercises a great influence upon its properties; yet, on account of lack of knowledge concerning such combinations, the cement chemist has been obliged to rely largely upon rule of thumb and cut-and-try methods in proportioning raw materials. Despite this fact, careful manufacturers have been able to produce uniform cement of excellent quality. This has been accomplished by the close observance of well-established facts concerning the effects produced by varying percentages of the elements and by giving especial care to the details of manufacture. A brief consideration of the effects of the main chemical elements and the reasons for controlling their proportions will now be given.

**330. The Proportioning of the Main Constituents.**—Lime, the element constituting five-eighths of Portland cement, should be very carefully proportioned with reference to the other ingredients. An excessively high lime contents tends to make the cement either unsound (i.e., to produce disintegration after setting), or slow setting with a high strength at an early age. On the other hand if the cement is underlimed it will probably be weak and either quick setting if it is underburned or slow setting if hard burned.\* A number of formulas for the regulation of the lime content have been proposed and are in use by cement chemists. S. B. Newberry, from the result of an elaborate set of synthetical analyses based upon the work of Le Chatelier, concludes that if proportions are stated by weight, the expression

$$\text{Lime} = \text{Silica} \times 2.7 + \text{Alumina} \times 1.0$$

is a safe rule for proportioning the raw materials commonly used in the United States provided they are finely ground and properly burned.\* It will be noted that this formula states only the proportions of the essential elements. It must not be concluded from this, however, that the proportions of the other elements are unimportant. To allow for the effects of magnesia and ferric oxide Eckel† modifies the rule thus

$$\text{Lime} + \text{Magnesia} \times 1.4 = \text{Silica} \times 2.8 + \text{Alumina} \times 1.1 + \text{Ferric oxide} \times 0.7.$$

Another criterion of more or less value is the hydraulic index which is the ratio  $\frac{\% \text{ silica} + \% \text{ alumina}}{\% \text{ lime}}$  in the finished cement. For most American Portland cements this index varies between .47 and .53.

From a consideration of the form of a hydraulic index it is evident that lowering the silica necessitates raising the alumina content in a cement. Roughly one can say that a high alumina content tends to render cement quick setting and strong at early ages, whereas a high silica content produces a slow setting cement of high strength. The relative proportions of these elements also affect the fusibility of the mixture in the kiln. On account of the marked influence which silica and alumina exert upon the time set, Meade has adopted an

$$\text{Index of activity} = \frac{\% \text{ silica} \dagger}{\% \text{ alumina}}.$$

\* See Newberry's publications in *Jour. Soc. Chem. Industry*, Vol. 16, p. 887; *Cement and Engineering News*, Nov. 1901; Taylor and Thompson's *Concrete, Plain and Reinforced*, Ch. VI.

† *Cements, Limes and Plasters*, p. 170.

‡ Meade's *Portland Cement*, p. 33; also *Engr. News*, Vol. 61, p. 374.



He states that this ratio is between 2.5 and 3.0 for most American cements and should always lie between 2.5 and 5. It is evident, however, from a consideration of the effects of lime, that the quick setting due to a high alumina content may be modified to some extent by increasing the lime content. Furthermore, the time of set may also be affected by iron and magnesia.

**331. Iron Oxide.**—Ferric oxide ( $\text{Fe}_2\text{O}_3$ ) exercises a very important influence upon the color of the cement. The pure white Portlands recently placed upon the market are made from materials containing very little of this element. Cements are also being made in which the alumina content is largely replaced by ferric oxide. Such cements have been advocated for use in sea water. Most cements, however, contain 2 or 3 per cent of this oxide. Some authorities, believing that iron oxide behaves like alumina in promoting compounds of silica and lime, insert it in the hydraulic index; thus the German specifications (1909) in defining Portland cement state that in the mixture of materials, the weight of lime to soluble silica plus alumina and iron oxide should not be less than 1.7 to 1.0. Small percentages of iron oxide render highly silicious raw materials easier to fuse, but a large iron content produces a hard clinker very difficult to grind.

**332. Magnesium Oxide.**—Magnesia has been the cause of much worry on the part of cement manufacturers and users, since the prevailing opinion has existed that a large proportion caused unsoundness. The results of experiments by Van Blaese (*Thonindustriezeitung*, 1899, p. 213) indicated that at least 5 per cent of magnesia was not hurtful, while those of Newberry showed that 9 per cent was not harmful to a well made of Portland cement. Butler \* reports a satisfactory English cement containing 5 per cent of magnesia. Dyckerhoff in *Cement Age*, Feb., 1909, reaffirmed a previous conclusion that over 4 per cent of this element is injurious.

P. H. Bates, of the Bureau of Standards, has conducted tests on cements varying in magnesia content up to 25 per cent.† These were prepared from mixtures of clay, dolomite, limestone and feldspar or kaolin burnt in a small gas-fired rotary kiln. The results of tests extending over a one-year period show that cements made with less than 7.5 per cent. magnesia are normal in setting, in soundness and in strength. Clinker containing less than 9.5 per cent. magnesia exhibits no abnormal properties.

Tests by Prof. E. D. Campbell at the University of Michigan show that free magnesia is a very undesirable constituent if more than 3 per cent is present in the cement. Neat prisms made from cements con-

\* *Portland Cement*, by D. B. Butler, p. 313.

† *Cement Age*, Cement Mill Edition, Mar., 1914.

taining 3 per cent free magnesia exhibited over 1 per cent expansion after thirteen years storage in cold water, although at an age of one year no unusual expansion was noted.

No opinion seems to be advanced indicating that a large percentage of magnesia is of value in a normally made Portland cement, consequently American specifications have limited its proportion to 5 per cent.

**333. Sulphur Compounds.**—The presence of sulphides in cement are undesirable, since they are liable to cause unsoundness. The sulphates of calcium, gypsum ( $\text{CaSO}_4 + 2\text{H}_2\text{O}$ ) and plaster of Paris ( $\text{CaSO}_4 + \frac{1}{2}\text{H}_2\text{O}$ ), are used in small percentages to retard the setting. They must be added subsequent to burning, however, in order to be effective. American specifications restrict the presence of sulphur trioxide ( $\text{SO}_3$ ) to 2.00 per cent.

**334. Alkalies.**—The alkalies ( $\text{K}_2\text{O}$  and  $\text{Na}_2\text{O}$ ) are found in very small amounts in most Portland cements. Their action is uncertain but, probably not beneficial.

**335. Carbonic Oxide.**—Since it is common practice to subject the clinker to aeration before final grinding, all cement contains some carbon dioxide. The percentage absorbed by the cement is dependent principally upon the completeness of exposure to the air as an experiment by Butler demonstrates. He placed a part of a sample of sound cement containing 0.25 per cent  $\text{CO}_2$  in a cask and part he spread out in a thin layer on a tray. At twenty-eight days the sample in the cask contained 0.57 per cent  $\text{CO}_2$  and the sample in the tray 1.82 per cent; the corresponding percentages after three years were 3.21 and 5.60. The tensile strength of the original sample showed a progressive increase during this time interval. From these results he concluded that the percentage of carbon dioxide in a cement is not an indication of free lime.\* American specifications prescribe a limit of 4 per cent for loss on ignition.

**336. The Constitution of Portland Cement.**†—Although it is a comparatively simple task to determine the percentages of the various chemical elements in Portland cement, the determination of the combinations of these elements which exist in the finely ground clinker is a problem which has baffled the most brilliant experts in physical chemistry. In this connection it should be remembered that the kiln temperature is only sufficient to start fusion and consequently complete solutions of all the elements are not obtained. Among the more prominent investigators of the constitution of Portland cement may be mentioned Le

\* Butler's *Portland Cement*, p. 317.

† For further information on this subject see *The Chemistry and Testing of Cement*, by Desch, *Portland Cement*, by Meade, also *Fused Portland Cement*, by C. Unger (translated article in *Cement and Engr. News*, Vol. 22, p. 192).

Chatelier,\* the noted French chemist; the Newberry Brothers;† Tornebohm,‡ the Swedish investigator; W. Michaelis, Sr.§ C.; Richardson;|| and Day, Shepherd, Rankin and Wright at the Geophysical Laboratory of the Carnegie Institute, Washington, D. C.¶

As a result of the extended researches at the Geophysical Laboratory in which optical, thermal, and chemical properties of a very large number of fused mixtures of pure lime, silica and alumina were investigated, it appears that there are five mixtures of minerals, any one of which may form the main constituents of a Portland cement clinker. In order of lime content in the raw mix these are: First, a high-limed mixture which solidifies after sintering to a mixture of  $\text{CaO}$ ,  $3\text{CaO} \cdot \text{SiO}_2$  and  $3\text{CaO} \cdot \text{Al}_2\text{O}_3$ ; second, "*presumably typical Portland cement clinker*" a mixture of  $2\text{CaO} \cdot \text{SiO}_2$ ,  $3\text{CaO} \cdot \text{SiO}_2$  and  $3\text{CaO} \cdot \text{Al}_2\text{O}_3$ ; third, a mixture of  $2\text{CaO} \cdot \text{SiO}_2$ ,  $3\text{CaO} \cdot \text{Al}_2\text{O}_3$  and  $5\text{CaO} \cdot 3\text{Al}_2\text{O}_3$ ; fourth, a mixture of  $2\text{CaO} \cdot \text{SiO}_2$ ,  $\text{CaO} \cdot \text{Al}_2\text{O}_3$ ,  $5\text{CaO} \cdot 3\text{Al}_2\text{O}_3$ ; fifth, a mixture tentatively stated to consist of  $2\text{CaO} \cdot \text{SiO}_2$ ,  $\text{CaO} \cdot \text{Al}_2\text{O}_3$  and  $2\text{CaO} \cdot \text{Al}_2\text{O}_3 \cdot \text{SiO}_2$ . The authors state that these relations will probably remain unaltered by the impurities ordinarily found in the clinker. "Of the effect of magnesia and iron oxide nothing definite is known. Neither seems to form solid solutions with  $2\text{CaO} \cdot \text{SiO}_2$ ,  $3\text{CaO} \cdot \text{SiO}_2$  or  $3\text{CaO} \cdot \text{Al}_2\text{O}_3$ . Nor at the present writing (1911) has any appreciable solid solution been discovered between  $\text{CaO}$ ,  $\text{Al}_2\text{O}_3$  and  $\text{SiO}_2$  compounds." ¶

Experiments by P. H. Bates on the constitution of cement clinker, in which burnings were made in a small rotary kiln, indicated that iron oxide entered the beta-orthosilicate ( $2\text{CaO} \cdot \text{SiO}_2$ ) and the 5 : 3 calcium aluminate ( $5\text{CaO} \cdot 3\text{Al}_2\text{O}_3$ ) but did not enter the tricalcic silicate ( $3\text{CaO} \cdot \text{SiO}_2$ ) or the tricalcic aluminate ( $3\text{CaO} \cdot \text{Al}_2\text{O}_3$ ). In the low-limed mixture, it formed dicalcic ferrite ( $2\text{CaO} \cdot \text{Fe}_2\text{O}_3$ ) and in the high-limed mixtures, opaque glass and magnetite.\*\* Bates has also shown that magnesia may enter into the beta-orthosilicate ( $2\text{CaO} \cdot \text{SiO}_2$ ) up to 6 per cent and into tricalcic aluminate ( $3\text{CaO} \cdot \text{Al}_2\text{O}_3$ ) up to 10 per cent. In cements carrying less than 7.5 per cent magnesia no separate magnesium compounds were discerned.††

**337. The Setting and Hardening of Portland Cement.**—If finely pulverized Portland cement clinker is tempered with the proper amount

\* *Constitution of Hydraulic Mortars*, by H. Le Chatelier (trans. by Mack).

† *Journal of the Society of Chemical Industry*, Nov. 30, 1897.

‡ *Ueber die Petrographie des Portland Zements*, Stockholm, 1897.

§ *The Constitution of Hydraulic Cements*, by W. Michaelis, Sr., pub. by *Cement and Engr. News*.

|| *Engr. News*, Vol. 52, p. 127.

¶ *Engr. News*, Vol. 65, p. 350; also *Jour. Industrial Chem.*, April, 1911.

\*\* Reported in *Cement and Engineering News*, Feb., 1913.

†† *Cement Age*, Cement Mill Edition, Mar., 1914.

of water, the resultant soft paste gradually loses its plasticity and becomes stiff and hard. When the paste has lost its plasticity and becomes sufficiently rigid so that it will withstand an arbitrarily defined pressure it is said to have set. Generally the period of setting is divided into two stages, the beginning and ending of the setting. The former is called the initial and the latter the final set. After the cement paste has attained final set, it further increases in rigidity, strength and hardness. To the latter transformation the term hardening is applied.

Since the setting and hardening of Portland cement is intimately connected with its constitution, there have been as many theories concerning the cause of setting as there have been regarding the constitution. Many of these theories have been advanced by the investigators mentioned in the preceding article.

A theory which has been accepted by many in recent years was first advanced by W. Michaelis, Sr., in 1893. His own statement of the theory follows.\*

"The formation of colloidal † calcium hydro-silicate, and to a lesser degree also the formation of colloidal calcium hydro-aluminate and calcium hydro-ferrite, is the only essential and important feature in the hardening process of all known calcareous hydraulic cements."

Michaelis maintains that when a finely ground hydraulic cement is gauged with water, calcium oxide, calcium aluminate, calcium sulphate, alkali silicate and possibly a little calcium ferrite are dissolved with the production of a supersaturated solution of these components. Calcium sulphate combines with tricalcium aluminate to form calcium sulpho-aluminate. Owing to the insolubility in lime water of the above mentioned compounds, clusters of imperfect crystals of needle or plate-like form are produced. Next the oversaturated solution coagulates about the cement grains, many of which have thus far not been attacked. There is thus formed a soft plastic hydrogel containing calcium hydrosilicate, calcium hydro-aluminate, calcium hydro-ferrite and a very small proportion of lime. Hardening of this soft hydrogel is brought about through the gradual absorption of water by the unattacked cement grains. This

\* *Cement and Engineering News* (trans.), Vol. 21, p. 299.

† There are a number of solids which when very finely pulverized and mixed with certain liquids in highly supersaturated solutions will form more or less rigid bodies by coagulation and subsequent desiccation. No crystalline structure is evinced during or after desiccation. On account of the glue-like properties which such substances exhibit in hardening, they have been named colloids. Some substances may form either colloids or crystalloids, depending upon the degree of supersaturation of the liquid solution. Colloidal solutions will not pass through a parchment membrane, whereas crystalloid solutions will. A colloidal aqueous solution is often called an hydrosol. When it becomes gelatinous it is termed an hydrogel. The boiling-point of an hydrosol is the same as the boiling-point of water.

withdrawal of water leaves the lime in the hydrogel envelope surrounding the cement grains. The hydrogel thus hardens and becomes impervious to the passage of water.

The strength of the hardening cement, Michaelis states, is increased by the crystallization of calcium hydrate, calcium hydro-aluminate, calcium hydro-ferrite, and to some extent by the crystallization of calcium sulpho-aluminate. These crystalloids are slowly formed from slightly oversaturated solutions, whereas the colloids are produced from highly supersaturated solutions. Although the crystalloids increase the strength of the colloids, they also decrease imperviousness on account of the boundary planes between the crystals and the colloids. The hardening of cement in air is hastened by the evaporation of a portion of the water. Owing to the evaporation of water during air hardening, however, more cement grains are imperfectly hydrated than in water hardening, and shrinkage of the colloid is very marked.

Since crystals are formed from solutions of low supersaturation, Michaelis reasons that the addition of gypsum which brings on crystallization, will therefore retard the setting process. This will appear evident when one considers that, owing to the low supersaturation of the initial mixture, some crystallization must take place before coagulation of the hydrogel.

In speaking of the causes of unsoundness, he states that on account of the non-homogeneity of Portland cement, arising from imperfect conditions of manufacture, there will always be more or less uncombined lime within the larger grains. The latter will eventually hydrate with an expansion which may or may not cause disintegration, depending upon the size of the grains and the elasticity of the hardened colloidal crust surrounding them.

Michaelis' theory has been partially verified by the microscope. Sections of hardened Portland cement paste have revealed the colloidal structure surrounding grains of unattacked clinker. In fact Stern \* estimated that only half of the cement grains are attacked by water in ordinary pastes of cement. That the attack of water is incomplete may be proved by holding the broken ends of cement briquettes in contact for several days after which cohesion will have taken place, or by regrinding said pastes and molding new specimens. It has been demonstrated that the powder formed by crushing and grinding neat cement briquettes has cementitious properties and briquettes made after a second regrinding have possessed a low strength.

Messrs. Klein and Phillips† after extensive microscopic investigations

\* *Chemiker Zeitung*, 1908, No. 47 and 85; *Stahl u. Eisen*, Vol. 28, p. 1542; *Mitt. Kgl. Materialprüfungsamt*, Vol. 27, p. 7, and Vol. 28, p. 173.

† *Technologic Paper* No. 43, U. S. Bureau of Standards, April, 1914.

on the hydration of Portland cement summarized their results as follows:

"The hydration of cements is thus brought about by the formation of amorphous hydrated tricalcium aluminate with or without amorphous alumina, the aluminate later crystallizing. At the same time sulphoaluminate crystals are formed, and low-burned or finely ground lime is hydrated. The formation of the above compounds begins within a short time after the cement is gauged. The next compound to react is tricalcium silicate. Its hydration may begin within twenty-four hours, and it is generally completed within seven days. Between seven and twenty-eight days the amorphous aluminate commences to crystallize and beta-orthosilicate begins to hydrate. Although the latter is the chief constituent of the American Portland Cements, it is the least reactive compound. The early strength (twenty-four hours) of cements is probably due to the hydration of free lime and the aluminates. The increase in strength between twenty-four hours and seven days depends upon the hydration of tricalcium silicate, although the further hydration of aluminates may contribute somewhat. The increase between seven and twenty-eight days is due to the hydration of beta-calcium orthosilicate, but here are encountered opposing forces, in the hydration of any high-burned free lime present and in the crystallization of the aluminate. It is to this hydration that the falling off in strength between seven and twenty-eight days of very high-burned high-limed cements is due, whereas the decrease shown by the high alumina cements is due to the crystallization of the aluminate. Finally, the iron in the cement is resistive to hydration and does not form any definite crystalline hydration products, but occurs as a rust-like material."

"The last statement is perfectly true for the *cements we have examined* and contains nothing derogatory to the so-called iron cement."

No opinion seems to be advanced indicating that a large percentage of magnesia is of value in a normally made Portland cement, consequently American specifications have limited its proportion to 5 per cent.

Further researches at the Bureau of Standards by Bates and Klein, (see *Technologic Paper* 78) indicate that hydrated dicalcic silicate hardens into a granular porous mass which is susceptible to mechanical breakdown when exposed to the crystallization of salt solutions; also, that hydrated tricalcic silicate forms a very dense structure of colloidal nature interspersed with crystals of lime hydrate, and consequently is subject to relatively large volume changes if alternately wetted and dried. It appears, therefore, that in good Portland cements these constituents should be well balanced with the dicalcic silicate slightly in excess. Under such conditions a maximum density of structure with a minimum coefficient of hygroscopic expansion will result.

## MODERN METHODS OF MANUFACTURE

**338. Growth and Importance of the Portland Cement Industry.**--On account of the many excellent properties possessed by Portland cement, its great field of usefulness, the widely distributed sources of the raw materials from which it is made, its cheapness, and the decline in the supply of timber, the Portland cement industry has had a marvelously rapid growth. Although the process of manufacture of this material was patented in 1824, by Joseph Aspdin, of Leeds, England, it was not until 1859 that any considerable quantity was used in England, and not until 1875 that any progress was made in the manufacture of this cement in the United States. During the latter year, the pioneer Portland cement plant in the United States was started by Messrs. D. O. Saylor, E. Rehrig and A. Woolever at Coplay, Pa. The plant bearing Saylor's name is still running to-day with a very much increased capacity. An estimate of the rapidity of growth and the economic importance of this industry may be formed by comparing the quantity produced in 1880—42,000 bbl.—with the output for the United States in 1916—91,521,198 bbl. The value per barrel at mill in 1880 was \$3, in 1916 it was \$1.06.

**339. Raw Materials.**—In general the calcareous ingredients are present in raw materials in the form of lime carbonate ( $\text{CaCO}_3$ ) and the silica and alumina are combined in the form of clay or other argillaceous ingredients. These essential ingredients are proportioned in accordance with Newberry's rule or some similar method.

Arranged in order of importance, the raw materials most commonly used in the manufacture of Portland cement and the parts of the United States in which they are employed are:

MATERIALS		WHERE USED IN MAKING PORT- LAND CEMENT
CALCAREOUS	ARGILLACEOUS	
1. Limestone	+ Shale or Clay	Widely used, Eastern N. Y., Mich., Ill., Ind., Cal., Ia., Kan., and Tex.
2. Limestone	+ Cement rock	Eastern Penn., N. J., Cal., and Kan.
3. Limestone	+ Blast furnace slag	Illinois, Ohio, Penn.
4. Marl	+ Shale or Clay	Central N. Y., Ohio, Mich., Ind.

Limestone for the manufacture of Portland cement should be soft and consist largely of calcite or calcite and clay. If 20 per cent or more of clay is present with the calcite it is called cement rock. The limestone should not contain over 5 or 6 per cent of magnesium carbonate and should be comparatively free from sulphur and quartz. According to Eckel \* phosphorus pentoxide ( $\text{P}_2\text{O}_5$ ) is also an undesirable element.

\* Eckel's *Cements, Limes and Plasters*, p. 389.

**Marl**, another calcareous substance much used in the wet process of manufacture, is a soft deposit found in the bottoms of shallow lakes, swamps and extinct fresh-water basins. It should conform to the restrictions placed upon limestone and be free from sand and gravel.

For origin and composition of clays, reference should be made to Art. 275, Ch. VIII.

With reference to the proportions of the constituents of clay suitable for Portland cement manufacture, Meade states that the ratio of the silica to the alumina content should be between 2.5 and 4 to 1, that there should not be more iron oxide than alumina, and that the alkalis and magnesia should each be less than 3 per cent.\* A considerable proportion of sand larger than a 100-mesh sieve renders the clay unfit for cement manufacture.

Blast furnace slag suitable for the manufacture of Portland cement should be basic in character. The analysis should conform roughly to the following: Two-fifths to one-half lime, one-third silica, one-eighth to one-sixth alumina plus iron oxide, magnesia less than 3 per cent. Calcium sulphide is considered undesirable.

The first step in the process of manufacture of Portland cement is the winning of the raw materials from nature. Hard raw materials are blasted, loaded on to small cars, and drawn to the cement mill. Soft materials like marl or clay are dug or excavated by steam shovel or dredge, depending upon the nature of the deposit. Such materials are often pumped directly to the mill. Slag is granulated into a sand-like substance by running the molten material from the blast furnace into a vat of water. It is then loaded on cars by clam-shell buckets and transported to the cement plant.

#### THE DRY PROCESS OF MANUFACTURE

**340. Preparation of Raw Materials.**—In general, only the comparatively dry raw materials, such as limestone and cement rock, limestone and shale or clay, and limestone and blast furnace slag, are used in the dry process of manufacture of Portland cement. The steps in the process of manufacture are: 1, crushing of raw materials; 2, drying; 3, grinding; 4, proportioning; 5, final pulverizing of raw materials; 6, burning; 7, cooling the clinker; 8, adulteration to retard set; 9, reduction of clinker to an impalpable powder; 10, seasoning of cement; 11, bagging. The order of the first four of these operations varies at different plants and it is dependent to some extent upon the character of the raw materials.

Crushing of the hard materials is largely done in gyratory crushers, although a few plants pass material from the quarry through toothed rolls. Generally, the material must be passed through a large and a small crusher

\* Meade's *Portland Cement*, p. 54.



in order that the requisite fineness for successful operation of the grinding mills may result.

Since it is necessary to have the raw materials in an approximately dry state before grinding, most of these materials must be passed through some sort of a drying apparatus. In most plants a dryer consists of a hollow steel cylinder about 50 ft. long and 5 ft. in diameter, revolving about its geometrical axis which is inclined at a small angle with the horizontal. The raw materials enter at the upper end and pass out at the lower end of

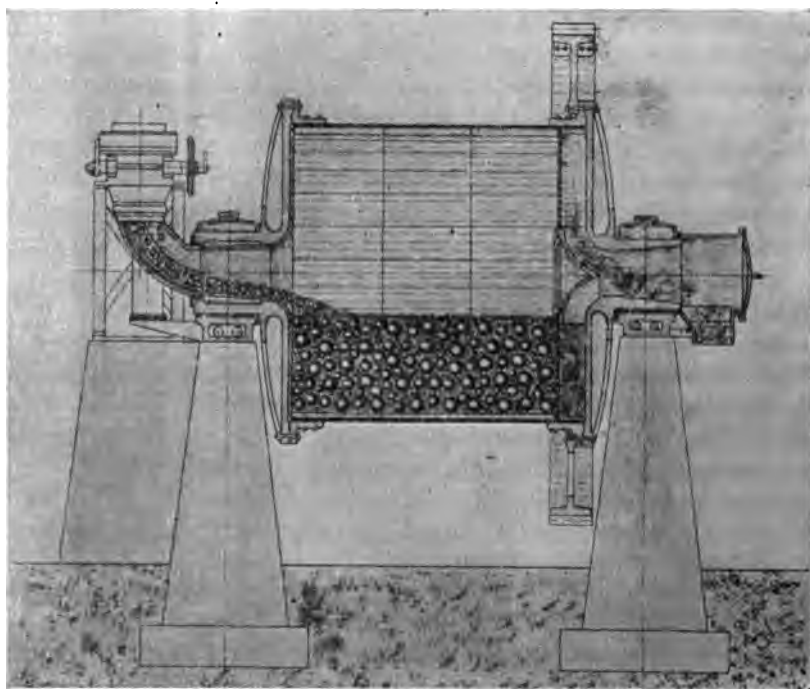


FIG. 1.—A Dry Grinding Preliminary. (Allis-Chalmers.)

the cylinder. The source of heat, which is commonly an attached furnace or waste gas from the rotary kilns, enters at the lower end and passes out at the upper. To increase the circulation of the materials through the hot gases, lugs which serve to elevate and scatter the charge, are riveted on the inside of the dryer.

**341. Preliminary Grinding.**—In order to secure proper combinations in the kiln between the lime, silica, and alumina, it is necessary to have the raw materials ground so finely that 95 per cent will pass a No. 100 mesh. It has been found economical to do this grinding in two stages, designated preliminary and final.

Preliminary grinding is quite extensively done in some type of ball mill. One of the latest forms of such machine is the preliminator shown in Fig. 1. The preliminator consists of a cylindrical steel drum lined with chilled iron or hard steel. To permit revolution of the machine the drum heads at either end are equipped with trunnions. The right end of the drum is also encircled by a large gear by means of which the mill is rotated. Through a hole in the left trunnion raw material is fed into the mill. Pulverizing is accomplished by the rolling and hammering received from six to eight tons of forged steel balls which range in diameter from  $2\frac{1}{2}$  in. to 5 in. The ground material passes through narrow slots in the vertical diaphragm shown near the right end of the mill and

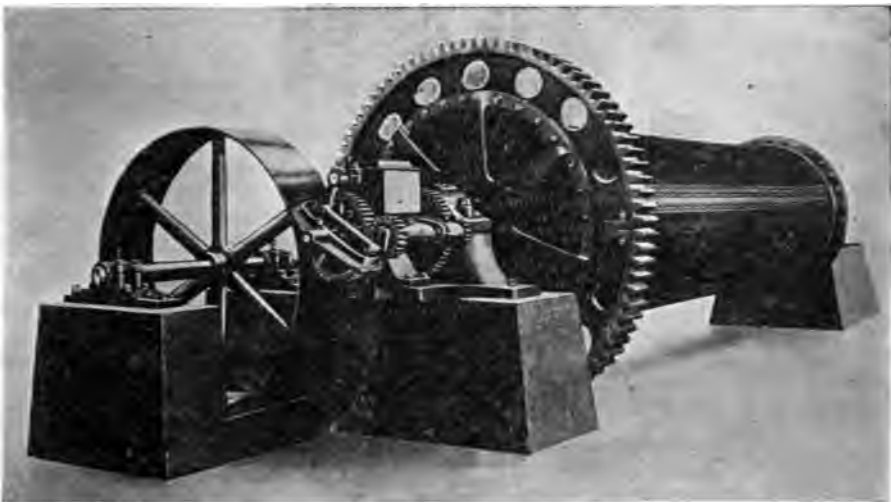


FIG. 2.—A Tube Mill. (Allis-Chalmers.)

is pitched through the right trunnion by a series of radial fins which are fastened to the center of the vertical diaphragm. Working on 3-in. limestone and shale, a preliminator 8 ft. in diameter and  $5\frac{1}{2}$  ft. long can produce about 50 bbl. per hour of material which will pass a No. 20 mesh.

**342. Proportioning.**—Since correct proportioning of the raw materials is of prime importance in securing a clinker of proper constitution, accurate automatic scales are installed for weighing the raw materials. This operation can generally be done to best advantage after the preliminary grinding of the raw materials; although at some plants, where the raw mix runs very uniform in character, the proportioning is done earlier in the process.

**343. Final Grinding**—At many plants the finishing stage in the grinding process is performed in a tube mill (Fig. 2). This mill is also steel-jacketed, cylindrical in shape and revolves about its geometrical axis.

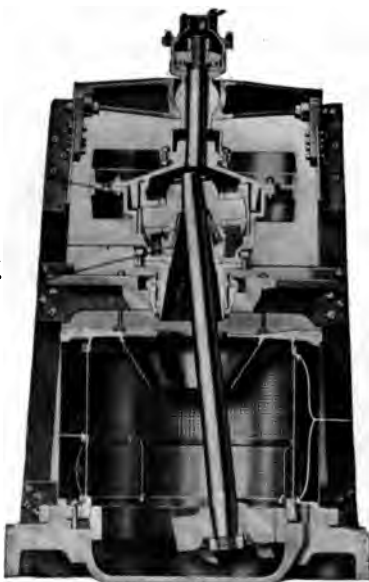
Commonly, such a mill is about 22 ft. long and 5 or 6 ft. in diameter. In order to form a surface with a high resistance to abrasion, the inside of the drum is lined either with trap rock, silix, or chilled cast iron.

The mix from the preliminary grinding machines is fed by a worm through the left trunnion of the mill. Pulverizing is accomplished by means of approximately 15 tons of flint stones, the largest of which is about the size of a goose egg. Lately the efficiency of the tube mill has been considerably increased by dividing the drum into two unequal compartments with a vertical diaphragm similar to that used in the

Pulley Rim on Lower Half Pulley Body eliminating shearing stress in bolts.

All wearing parts run in oil maintained by Oil Retainer increasing their life and reducing oil consumption.

Double Screen allowing total height of 36 inches. Lower half takes most wear, minimising screen maintenance.



By removing 4 bolts, Upper Cross Bar, Pulley Body, and Shaft can be raised, allowing easy replacement of Roll and Die.

Lower Journal Bearing separate from Pulley Body, decreasing cost of wearing parts.

Increased screening area allows freedom of discharge from mill, making it practically dustless and producing a larger output or a finer material.

FIG. 3.—A Giant Griffin Mill with Improvements.

preliminator. In this type of machine the raw mix is fed through the left trunnion into the longer compartment and ground by means of flint stones. It then passes through the diaphragm into the short finishing compartment where it is acted upon by small metal balls or slugs. From the finishing compartment the powdered mix passes through a second diaphragm into the right trunnion, whence it is discharged onto a belt conveyor. A 6×22-ft. tube mill will grind about 5 or 6 tons of raw material per hour, or it will reduce about 24 bbl. of ground clinker to cement in the same interval of time. By making use of a short finishing compartment in its tube mills, the Allis-Chalmers Company claims that an increase in both quantity and fineness of product can be secured at a somewhat greater expense for power.

Fig. 3 shows a sectional elevation of a Griffin mill which is used in

many plants as a substitute for the tube mill. In this machine the material enters the pan at the bottom and is forced upward between the circular die and revolving ring. The latter is rotated at approximately 150 r.p.m. by means of the pulley and universal joint at the top of the shaft, so that there is developed between the die and ring a very large centrifugal force which rapidly pulverizes the material. A current of air carries the fine material upward through screens in the top of the pan while the coarse material falls to the bottom and is reground.

Another type of grinding mill often used on raw material is the Lehigh-Fuller, shown in Fig. 4. In this mill four 12-in. steel balls are pushed around an annular die by means of horizontal radial arms set 90 degrees apart on the vertical shaft. Since the shaft runs at 160 r.p.m. the balls exert a large force against the die. The materials are fed to the mill from a hopper on top, which is provided with a feeder operated from the mill shaft. The material is discharged by the feeder between the balls and the die and is thus reduced to an impalpable powder. By means of a centrifugal fan operating in the separating chamber just above the pulverizing zone, the powdered material is drawn into the separating compartment. The ground material is drawn through the circumferential screens surrounding the separating chamber and forced through the discharge spout by a lower fan, shown below the ball in the figure.

There are several other types of grinding machines, the Maxecon, Huntington, Raymond, Sturtevant Ring-Roll mill, etc., but lack of space prohibits a discussion of them.

**344 Burning.**—The purpose of burning the raw mixture is to secure a union of the different constituents in the form of mineral compounds,

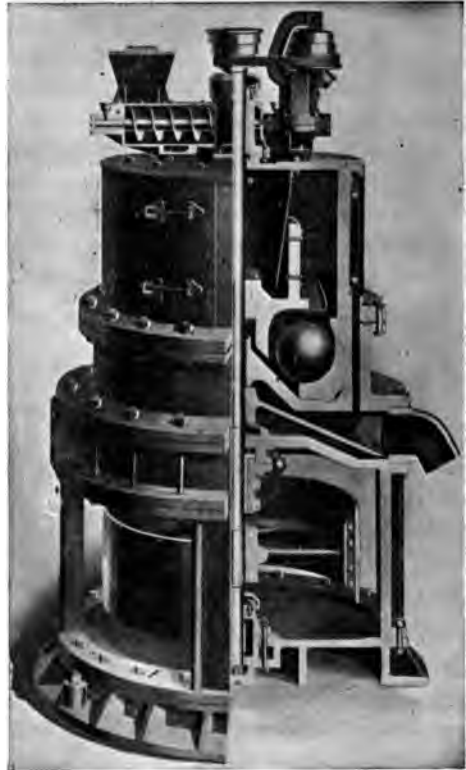


FIG. 4.—A 42-in. Fuller Mill Equipped with Fan Discharge.

primarily the formation of silicates of lime and alumina as we have seen in Art. 336.

After the raw materials have been proportioned, intimately mixed, and very finely ground, the powdered product is conveyed to kilns to be burned. Formerly the vertical intermittent type of kiln, somewhat like that used in the production of natural cement, was employed to burn Portland cement. In Europe, use is still made of this type, and in Germany the Hoffman ring kiln, is quite extensively employed. However, in the United States the continuously operated rotary kiln is favored to the exclusion of all others.

From Fig. 5 one can obtain a notion of the appearance of a rotary kiln. It consists of a cylindrical jacket made of riveted steel plates lined with

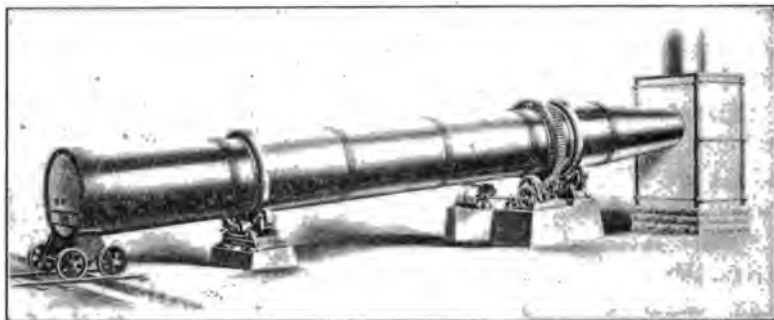


FIG. 5.—A Rotary Kiln.

refractory fire bricks. The lower end of this cylinder is covered by a detachable hood provided with two openings. Through one of these openings is passed a nozzle for the admission of fuel. The fuel most commonly employed is powdered coal. In order to introduce the coal into the kiln and to secure both rapid and complete combustion, it must be so finely pulverized that 95 per cent will pass a No. 100 sieve. The coal is blown through the nozzle by an air blast. The second opening in the hood is provided to enable the operator to observe the interior of the kiln during calcination. The steel jacket is surrounded by two or more heavy steel tires, by means of which it is rotated on friction roller bearings. These bearings are so adjusted that the axis of the kiln has an inclination with the horizontal of about  $\frac{1}{4}$  in. per foot. By thus inclining the axis, the material is slowly moved downward from the upper end as the kiln is rotated. Rotation is produced by a motor placed near the center and geared to a girth gear attached to the jacket. The upper end of the kiln enters a brick flue from which the products of combustion escape to the stack. Passing through this flue is an inclined spout which discharges the finely powdered raw material into the kiln.

Soon after the entrance of the material, it begins to ball up into small marble-like shapes. During the first half of the passage toward the hood any entrained water is evaporated and the material is heated to a temperature sufficient to expel carbon dioxide ( $\text{CO}_2$ ) from the limestone. By the time the clinker has reached within a few feet of the lower end of the kiln its temperature has risen to 1400 or 1500° C., all carbon dioxide sulphur, and organic matter have been expelled, and the little soft yellowish-brown balls have now partially fused into hard, greenish-black clinker.\* At many plants the degree of calcination is left to the skill of the burner, who regulates the speed of rotation of the kiln so that the clinkering zone is kept back a few feet from the discharge end. He is able to judge of the position of this zone by greenish flame which is emitted when the material begins to burn and form clinker. Under ordinary conditions a speed of 30 or 40 revolutions per hour secures the requisite degree of calcination. At the end of about one hour the burning process is completed and the clinker falls out of the kiln through a trap in the lower side of the hood, whence it is conveyed to the cooler. The clinker is quite irregular in shape and varies from the size of a hen's egg down to a buckshot. It is very hard, has more or less vitreous luster, and is generally black or greenish-black in color.

Most modern kilns are from 100 to 150 ft. long and from 6 to 9 ft. in diameter. At present the tendency is toward the use of longer kilns. The capacity of kilns of the sizes mentioned will vary from 400 to 800 bbls. per day. In producing a 376-lb. barrel of Portland cement, from 600 to 700 lb. of raw material and 80 to 120 lb. of coal is consumed.

**345. Grinding of the Clinker.**—To facilitate grinding, the clinker is now cooled by spraying with a water jet and passing through a cooler. Frequently the cooler consists of a vertical or horizontal steel cylinder equipped with devices for agitating the material. As the clinker passes through the cooler, it meets a forced air draft, which rapidly lowers its temperature. At some plants the clinker is cooled out of doors. After cooling, about 2 per cent of gypsum is added to lessen the time of set of the resulting cement. At many plants the adulteration is done after the clinker has been through the ball mill. In grinding the clinker the same kind of machinery is generally used as is employed in pulverizing the raw materials. Whatever grinding machinery is used, the resultant fineness must be such that 78 per cent will pass a sieve with 200 meshes per linear inch.† On account of the fact that a finely ground cement will make a stronger mortar than a coarsely ground one, many plants endeavor to secure a degree of fineness considerably in excess of

\* For further information concerning the chemical changes in a rotary kiln, see R. K. Meade's *Portland Cement*, pp. 176 to 199.

† See *Standard Cement Specifications of the A. S. T. M.*

the above figures. At present, however, the economical limit to which fine grinding may be carried seems to be about 85 per cent through a No. 200 sieve.

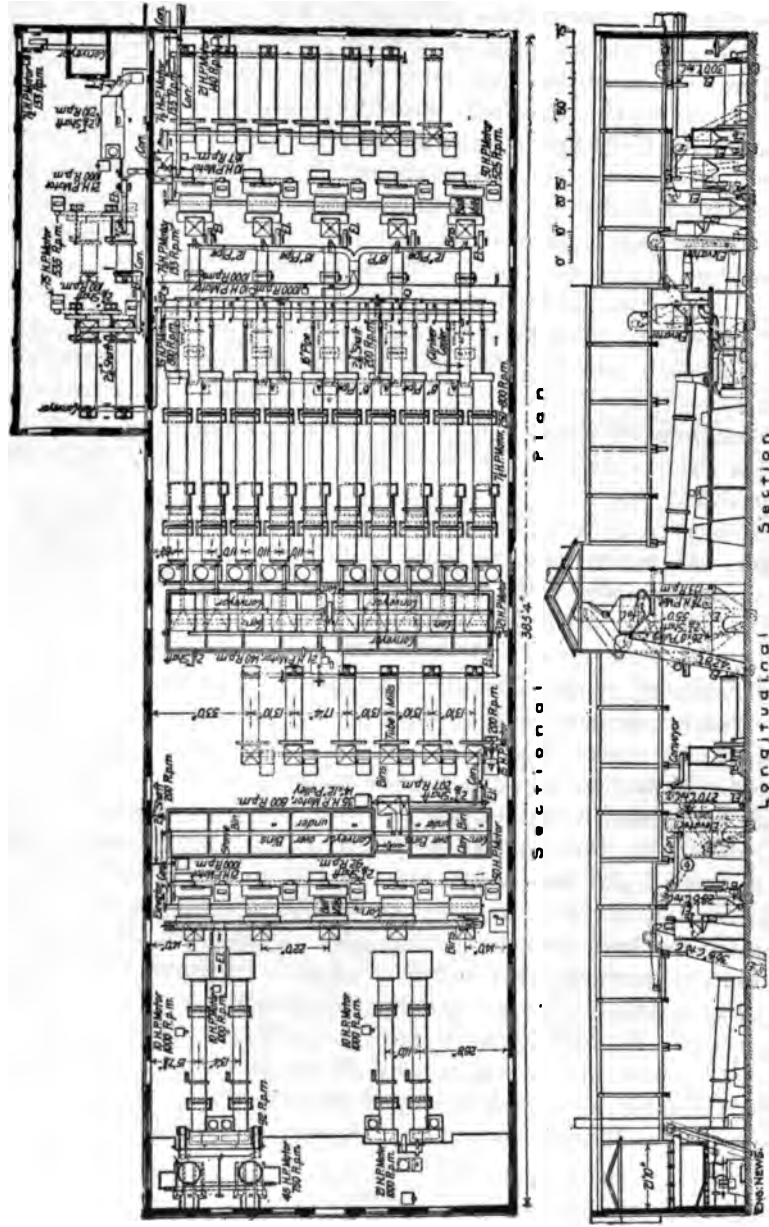


Fig. 6.—Plan of Hudson Portland Cement's Company's Plant. (*Engr. News*, Vol. 50, p. 70.)

**346. Storage and Bagging of Cement.**—From the grinding mills a conveyor carries the cement to the storage bins, in which it is generally kept for a few weeks before being bagged for shipment. This seasoning period seems to improve the quality of the cement. In support of this statement, it seems quite possible that unburnt lime might be sealed within the clinker during the burning period and be liberated during the grinding process. Upon exposure to the air such particles of lime would absorb carbon dioxide ( $\text{CO}_2$ ) and become calcium carbonate ( $\text{CaCO}_3$ ). This substance is not affected by the addition of water, is inert during the setting of the cement, and, therefore, does no harm.

In accordance with the demand of the trade the cement is conveyed from the storage bins to the packing house. Here it is automatically weighed and packed by machines sometimes in wooden barrels containing 376 lb., net, but more frequently in cloth sacks which hold 94 lb. net. The cement is now ready for shipment.

**347. Plan of Cement Plant.**—In order that an idea of the arrangement of a cement mill may be gotten, Fig. 6 showing a plan and vertical section of the Hudson Portland Cement Company's plant has been inserted.

#### THE WET PROCESS OF MANUFACTURE

**348. General.**—By far the larger portion of the cement manufactured in the United States is made by the dry process. Under favorable conditions, however, the wet process is successfully and economically employed.

The raw materials most commonly used in the wet process are marl and clay. Preparatory to mixing the materials, the marl is often screened and pumped, in the form of thin mud, from the deposit directly into large storage basins situated near the kilns. The clay is dried, for convenience in calculating the mixtures, and pulverized in an edge runner or similar mill, Fig. 1, Ch. VIII. Then, proper quantities of the two materials determined by a chemical analysis are weighed out and mixed by passing through a pug mill. This wet mixture, or slurry, from the pug mill consists of about two-thirds water and one-third marl and clay. The slurry is next pumped into large vats, in which it is continually agitated to maintain the uniformity of the solution. From these vats samples are taken for analysis; and, if necessary, additions of marl or clay are made until the desired composition is obtained. The slurry is now pumped into especially constructed rotary kilns and burned. The succeeding stages in the wet process of manufacture are similar to those described under the dry process.

**349. Comparison of Wet and Dry Processes.**—The chief advantage possessed by the wet process is the well-regulated control which is obtained



over the raw mixture. On the other hand, the wet process requires about one-third more fuel per barrel of cement than the dry process and the kiln capacity is about 25 per cent less than in the dry process. Although this increase in the cost of production by the wet process is partially offset by the higher cost of grinding the raw materials in the dry process, yet it is only when the raw materials can be gotten under very favorable conditions that a wet process cement can be made at a price which can compete with a cement made by the dry process.

#### EFFECTS ON PROPERTIES DUE TO CONDITIONS OF MANUFACTURE OR TESTING

**350. Conditions Affecting Soundness.**—Although some of the effects of the chemical elements present in Portland cement have already been referred to it will not be amiss to recall them in considering the properties affected.

The cause most commonly ascribed for unsoundness in Portland cement is the hydration of uncombined lime incased within the cement particles. High burned, coarsely ground, free lime hydrates slowly, but ultimately with sufficient violence to endanger the integrity of the surrounding mortar. Exposed, finely ground, free lime, in small percentages at least, will hydrate before the cement sets and produces no injurious effect. The presence of uncombined lime may be the result of either underburning the clinker or overliming the mixture before burning. Oftentimes freshly ground cement will be unsound due to the presence of uncombined lime which may be partially exposed in the grinding process. By allowing the cement to aerate for two or three weeks, thus allowing the lime to hydrate, it is often possible to overcome unsoundness.\*

Other chemical elements which may produce unsoundness are magnesia and the alkalies. In most cements the proportions of these elements are well within the danger limit.

It is probable that the action of the retardant assists in overcoming unsoundness, since it tends to hold the mixture in a plastic state and permit the lime to slake. Meade cites several examples of unsound cements which were rendered sound by adding from 0.5 to 3.0 per cent of plaster of Paris.†

Fine grinding of both raw materials and the clinker are very essential if a sound cement is to be secured. Fine grinding of the raw materials makes possible the production of a more homogeneous mixture before

\* For example see *Proc. A. S. T. M.*, Vol. 3, p. 376.

† *Portland Cement*, p. 474. See *Free Lime in Portland Cement* by Kiefer, *Chem. Engr.*, Vol. 15, p. 219; also *Soundness Tests of Portland Cement*, by Taylor, *Proc. A. S. T. M.*, Vol. 3, p. 374.

turning so that a uniform distribution of the lime content may obtain. It has also been shown that coarsely ground cements which are unsound in the accelerated test may often be rendered sound by fine grinding.\* Remembering that the addition of free lime in small percentages does not affect the soundness of a normal Portland cement, it seems evident that the coarser grains of cement may imprison minute particles of uncombined lime which do not become hydrated until after the cement has set. The expansion, which then occurs due to the crystallization of calcium hydrate, produces disintegration. This is entirely in accord with Michaelis' views on hardening and unsoundness (Art. 337). The reasonableness of the above explanation of the action of the coarser particles in promoting unsoundness is made more evident by the experiments of

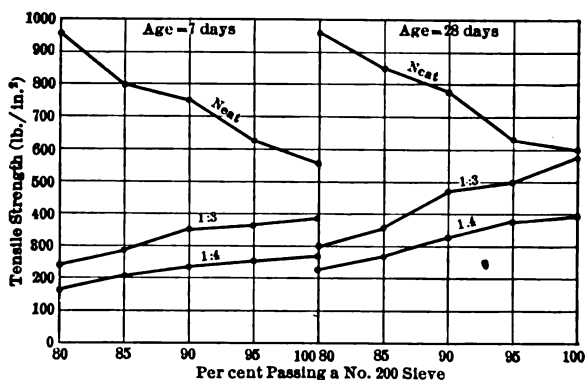


FIG. 7.—The Tensile Strength of the Same Cement when Ground to Different Degrees of Fineness. Each point represents the average of five briquettes. (Meade in *Proc. A.S.T.M.*, Vol. 8, p. 412.)

rinkley.† The results of his tests show that the particles of a cement passing a No. 100 and caught on a No. 200 sieve may have some hydraulic properties but that pats made of them break down entirely when subjected to the A.S.T.M. accelerated test. (Art. 419.)

**351. Conditions Affecting Strength.**—Recapitulating what has already been stated concerning the effect of various chemical elements upon the strength of Portland cement, we will remember that either a high lime or high alumina content tends to make the cement strong at an early age. Gypsum and plaster of Paris in small percentages also tend to increase the strength of Portland cement, but when present in quantities larger than 3 per cent these substances produce variable effects.‡ However,

\* *Portland Cement*, p. 472.

† *Engr. Record*, Vol. 61, p. 212.

‡ See results of experiments given in Eckel's *Cements, Limes, and Plasters*, pp. 6-544; also a paper by P. H. Bates in *Proc. A. S. T. M.*, Vol. 15, p. 126.

it is certain, as has been mentioned before, that the combinations formed by these elements, not the percentages in which they are found, are the factors influencing the strength.

In addition to the effects of the chemical constituents, the strength of cement is greatly influenced by the degree of burning, the fineness of grinding and the aeration it receives. If underburned the cement is liable to be deficient in strength. Assuming that the clinker has been

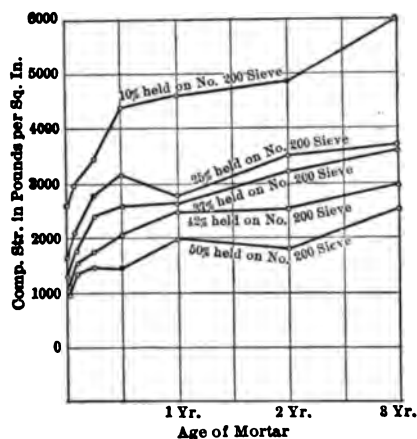


FIG. 8.—The Effect of Fine Grinding of Portland Cement on the Compressive Strength of 1 : 3 Mortar. (Tetmajer.)

the 200-mesh sieve had but little cementing power.† Since, however, the same cement was subjected to the same method of regrinding in each of the tests illustrated, siftings on the given sieves furnished indices of the fineness of the particles.

In Bates' experiments the effect of fineness of grinding on the compressive strength of 1 : 1½ : 4½ concrete was also determined. Ten brands of cement, varying in fineness from 75.4 to 82.2 per cent. passing a 200-mesh sieve, were used as received and also after being reground so that the fineness of the individual brands varied from 86.8 to 92.7 per cent passing the same sieve. At an age of twenty-eight days the concrete made from the finer cements exhibited an average strength of 28 per cent greater than the concrete made from the normal cement. At ninety days the concrete made from the finer cement averaged 17 per cent. more strength.

\* *Proc. A. S. T. M.*, Vol. 15, p. 126.

† See also Taylor's *Practical Cement Testing*, p. 107; *Proc. I. A. T. M.*, 6th Congress, 15, and 15.

‡ *Engr. Record*, Vol. 58, p. 181.

properly burned, increasing the proportion of flour or very fine particles will cause an increase in the mortar strength. Sometimes the neat strength is decreased by increasing the percentage of flour, but experiments by P. H. Bates \* show that this is not, in general, true. The effect of fine grinding is well shown in Figs. 7 and 8.† In interpreting these diagrams one must bear in mind that neither the 200-mesh sieve nor any other which has ever been made is fine enough to separate the flour possessing the maximum cementing power. In fact, Meade has shown that clinker ground so that it would barely pass

From the above it appears evident that a well-burned, finely ground cement can carry a greater proportion of sand than a more coarsely ground cement and will be more economical, provided the cost of the additional grinding does not offset the advantage derived.

The amount of "seasoning" or aeration which the cement has received subsequent to final grinding also exercises an important effect upon its strength. In general, it may be said that exposure of cement to the air is beneficial only when it is received in an unsound condition. Further seasoning than that required to remove unsoundness is detrimental to strength. The results of tests made by W. P. Taylor \* on cement stored in a cloth sack in his laboratory show that storage beyond three months was detrimental to the strength of both neat and mortar briquettes.

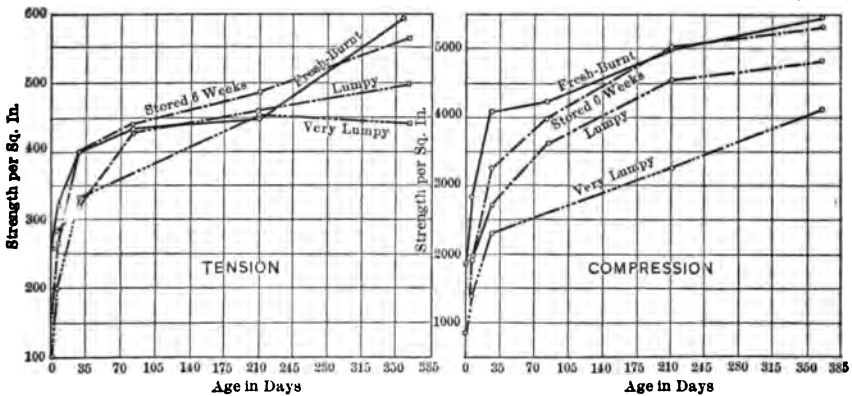


FIG. 9.—Effect of Long Storage on the Strength of 1 : 3 Portland Cement Mortar. (Tetmajer's *Communications*, Vol. 7, p. 15.)

(Briquette is the term applied to the standard form of tensile test-piece for cement mortars shown in Fig. 11, Ch. XII.) Both kinds of specimens showed the greatest effects at early ages, the mortar briquettes being weakened more than the neat by the aeration. Under the caption, "The Relative Value of Fresh and Caked Cement," *Engr. News*, Vol. 55, p. 67, may be found the results of tests by a number of different investigators which indicate, in the main, that fresh cement is preferable. A number of tests made at Stevens Institute of Technology † under artificial aging conditions also point to the conclusion, that Portland cement decreases in strength if aerated for a considerable length of time. Fig. 9 shows results obtained by Tetmajer.

The effect of the percentage of mixing water on the strength of 1 : 3 standard sand mortars is shown in Fig. 10. From these results it appears

\* *Practical Cement Testing*, p. 104.

† *Stevens Indicator*, Vol. 26, p. 158.

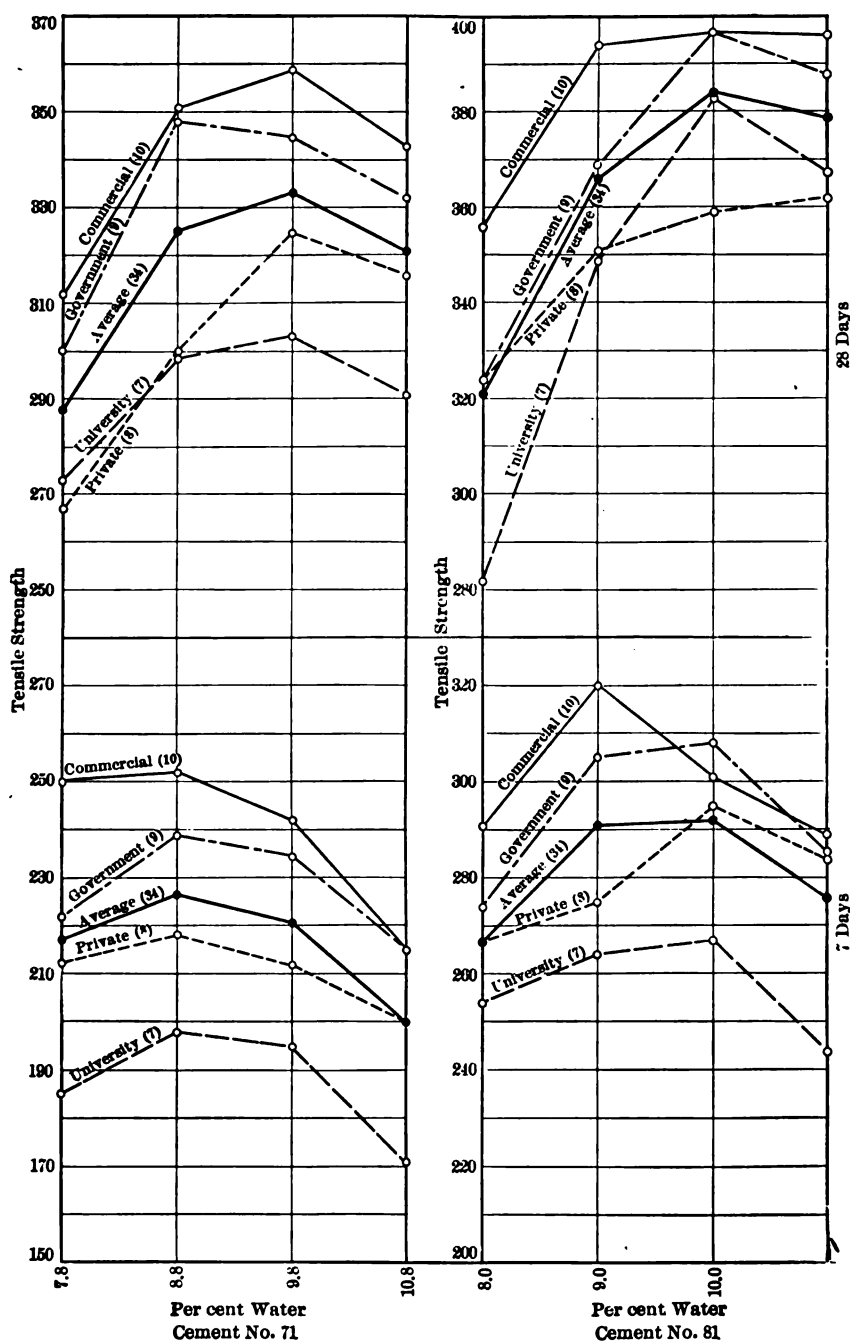


FIG. 10.—The Average Tensile Strengths of 1:3 Standard Sand Mortars at Seven and Twenty-eight Days, Mixed with Four Different Percentages of Water. (From Report of Joint Conference on Uniform Methods of Tests and Standard Specifications for Cement, Apr. 28, 1915. The figures in parentheses indicate the number of laboratories participating in the tests. About 2200 tests are represented.)

that either very dry or very wet mixes, which are molded in accordance with the A.S.T.M. method, are less strong than the mixtures of medium consistency.

If briquettes are removed from the water bath before testing a considerable effect, dependent upon the time they are allowed to dry, may be produced upon their strength. Fig. 11 has been compiled from the results of experiments by A. J. Barclay,\* on four different brands of cement. Similar tests have been reported by Prof. J. L. Van Ornum in *Eng. News*, Vol. 51, p. 24, and by Prof. R. P. Davis in *Eng. News*, Vol. 61, p. 581.

**352. Conditions Affecting the Time of Set.**—As we have already stated the theory of setting and hardening of Portland cement is not completely

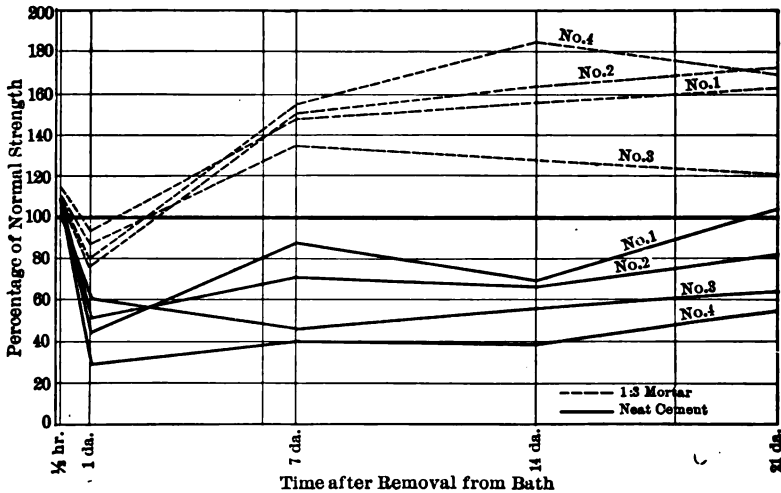


FIG. 11.—Effects of Drying on the Tensile Strengths of Neat and 1 : 3 Standard Sand Mortars. Age at removal from bath was twenty-eight days. Each result represents four tests.

known; it is not possible, therefore, to formulate definite rules for the effect which various chemical elements will produce upon the setting of cement. However, inasmuch as this property is very difficult to control and has such an important effect upon the value of the cement, certain commonly observed facts and the results of trustworthy experiments should be stated.

The things which influence the setting properties of the cement are its composition, the percentage of retardant, degree of calcination, fineness of grinding, aeration subsequent to grinding of clinker, percentage of water used in gauging the paste, the temperature of the mixing water

\* Thesis, University of Wisconsin, 1912.

and cement, the humidity and temperature of the moist closet or of the atmosphere in which the cement paste is placed, and the amount of manipulation the paste receives.

The effect of lime, silica and alumina in controlling the set have already been referred to in Art. 330. In addition to properly regulating the proportions of the above elements, or adjusting hydraulic and activity indexes which amounts to the same thing, some provision must be made by the manufacturer to increase the time of setting of freshly ground cement so that it will be sufficiently slow-setting for use in construction. This is commonly done by mixing gypsum ( $\text{CaSO}_4 + 2\text{H}_2\text{O}$ ) or plaster of Paris ( $\text{CaSO}_4 + \frac{1}{2}\text{H}_2\text{O}$ ) with the clinker before final grinding, or by adding one of these compounds just after the clinker has received preliminary grinding. Although these methods are manifestly imperfect since it is impossible to obtain an absolutely homogeneous mixture with either, yet they are the best now devised. (The addition of gypsum before calcination causes it to decompose into lime and sulphur trioxide. Since the latter is liberated in the kiln the resulting effect on the time of set is nil.)

Experiments by Le Chatelier and later ones by Meade and Gano,\* have shown that anhydrous calcium sulphate, plaster of Paris or gypsum may be used as the retardant. The experiments of Meade and Gano, however, indicate that increasing any one of these elements up to 2 or 3 per cent (the limit will vary with the chemical composition of the cement) retards the set but further additions of plaster of Paris cause the setting time to decrease. The introduction of 10 to 20 per cent of plaster of Paris will generally cause the cement to become quick-setting again. The latter effects were not observed in the tests made with gypsum ( $\text{CaSO}_4 + 2\text{H}_2\text{O}$ ) or dead burned gypsum ( $\text{CaSO}_4$ ).

In addition to the above-mentioned elements, small percentages of calcium chloride † and sodium carbonate also ‡ have a marked influence upon the setting properties of cement. Nihoul and Dufosseze showed that strontium sulphate, barium sulphate, calcium sulphate, and calcium aluminate in small percentages also effected a rapid increase in set.

Often an underlimed cement will become quick-setting after seasoning. This fault can be overcome by increasing the lime content in the raw materials or the remedy mentioned below may be applied to the cement. Examples of cements which become slower setting with age are common, and some cases have been cited where cements slow-setting when fresh have become quick-setting and then slow-setting after aging for some time.

\* *Chemical Engineer*, Vol. 1, p. 92; see also the tests of Nihoul and Dufosseze, abstracted in *Jour. Soc. Chem. Industry*, Vol. 21, p. 859.

† *Engr. News*, Vol. 53, p. 13.

‡ *Concrete-Cement Age*, Nov. 1912, p. 68.

Quick-setting may often be avoided by adding to the cement, 1 or 2 per cent of hydrated lime or the fraction of a per cent of plaster of Paris.

The fineness to which a cement is ground produces an effect upon its time of set as the results in Table 1 \* indicate. In general, it may be said that the more finely the cement is ground the more rapidly will it set. However, the relation between time of set and fineness as measured by

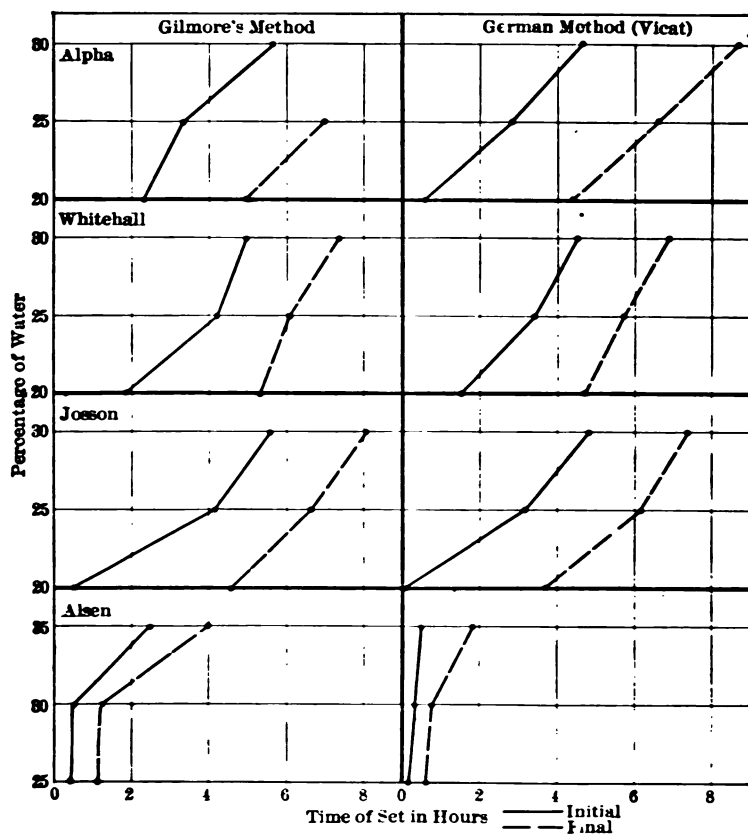


FIG. 12.—Effect of Percentage of Mixing Water on the Time of Set of Portland Cements, as Determined by the Gillmore and Vicat Methods.

the 200-mesh sieve is not rectilinear as inspection of the table will show. The results also indicate that grinding to a fineness of 90 per cent through the No. 200 sieve would not decrease the time of set below the customary limits.

The effect of the percentage of mixing water upon the time of set is well illustrated in Fig. 12 † which also furnishes a comparison of the

\* From a paper by Meade, *Proc. A.S.T.M.*, Vol. 8, p. 410.

† From *Tests of Metals*, 1901, p. 492.



results gotten with the Gillmore and German methods. The latter is essentially the same as the Vicat method commonly employed in America. (The Gillmore and Vicat methods are described in Art. 425). Another comparison of the Vicat and Gillmore methods is afforded in Table 4.

TABLE 1.—INFLUENCE OF FINE GRINDING UPON THE INITIAL SET OF CEMENT (MEADE)

Per Cent Passing a No. 200 Sieve.	SETTING TIME IN MINUTES OF CEMENT No.							
	1	2	3	4	5	6	7	8
75	255	105	120	240	240	200	100	115
80	246	106	115	200	210	190	100	105
85	192	100	100	180	110	175	90	100
90	75	100	95	115	55	100	80	75
95	12	22	60	60	15	25	25	30
100	2	6	35	30	5	2	5	10

The influence of temperature upon the time of set is shown in Table 2. Cements stored in warm rooms will, in general, be quicker setting than those stored in a cold atmosphere. Cold mixing water retards set while warm water accelerates it. For the range of temperature ordinarily met in the laboratory say 65° to 75° F., the effect is not very marked. However, due consideration of the influence of temperature should be given

TABLE 2—INFLUENCE OF TEMPERATURE ON THE SETTING OF PORTLAND CEMENT †

Sample No.	INITIAL SET IN MINUTES. TEMPERATURE ° F.				FINAL SET IN HOURS. TEMPERATURE ° F.			
	100	80	60	40	100	80	60	40
1	1½	4	6	13	1½	1½	2	2½
2	3	5	6	8	1	1½	1½	2½
3	4	10	15	20	½	¾	1½	6½
4*	5	9	15	30	½	¾	1	6
5	6	10	14	25	1	1½	2	2½
6*	7	12	15	20	1½	2	2½	2½
7*	9	10	15	17	3½	6	7	12
8	10	15	35	40	¾	1	1½	1½
9	11	15	20	57	3	5	6	10
10	11	13	15	30	2½	3	3½	6
11	19	32	60	120	3	6	7	15
12	15	35	70*	360	3½	6	7	22

\* Contain a considerable admixture of Kentish Rag.

† From Butler's *Portland Cement*, p. 267.

in reporting on cements which are quick-setting in a hot laboratory but which will be used in a colder atmosphere.

Cements exposed to a thoroughly saturated atmosphere will set much more slowly than those exposed to a dry atmosphere. If, however, a considerable proportion of moist  $\text{CO}_2$  is present in the air, the experiments of Gadd \* seem to indicate that the setting time will be greatly reduced.

By lengthening the time of mixing and by prolonged troweling of the surface mortars it is also possible to considerably delay the time of set.

**353. Conditions Affecting Fineness.**—The percentage of flour contained in a cement is principally dependent upon a number of variables in the method of manufacture. The chemical composition and the degree of calcination influence the hardness of the clinker and consequently affect the fineness to which the clinker is ground. Clinker high in iron or silica is apt to be hard and difficult to grind. The same is true of a hard-burned clinker. It does not always follow, therefore, that a difference in fineness indicates the relative quality of two cements since the one more finely ground may have been underburned. Furthermore, the fineness will be influenced by the time of grinding and the character of the pulverizing machinery employed in grinding.

To some extent seasoning also affects fineness. It has been found that cement becomes slightly finer with age provided it does not absorb too much moisture. This is probably due to the decrepitation of the coarser grains resulting from hydration of the embedded lime particles.

In testing the fineness of cement, the precaution which should be employed and the errors in sieves are mentioned in Art. 409 and 400.

**354. Conditions Affecting Specific Gravity.**—It is probable that no property of cement has had its importance more overrated than specific gravity. As a matter of fact, in the majority of cases, the specific gravity affords little if any information concerning the relative value of two cements made from different materials, unless the average specific gravity of each brand is known. The test is chiefly used to detect abnormal conditions in a brand of known specific gravity.

The detection of adulteration by this test is dependent upon the specific gravity of the adulterant and upon the proportion used. A simple computation reveals that a clinker having a specific gravity of 3.15 may be adulterated with 14.3 per cent of limestone having a specific gravity of 2.8 before the specific gravity of the mixture will be reduced below the requirement of the specifications, 3.10. If, instead of limestone, a blast-furnace slag or natural cement with a specific gravity of 3.0 be employed, 6.7 per cent of the adulterant may be used before the specific gravity of the adulterated cement is reduced 0.01. Furthermore, it is

\* See *Cement Concrete Age*, Cement Mill Section, Feb., 1914; also paper by G. M. Williams, *Proc. A.S.T.M.*, Vol. 14, p. 174.

permissible under the above methods of testing to ignite the sample if its specific gravity falls below 3.10. Experiments have shown that this procedure will raise the specific gravity of many adulterated mixtures considerably above the specified limit. So it is evident that, although adulteration lowers specific gravity, a low result is not necessarily a sign of adulteration, nor is a high value an indication of the absence of it.

Long seasoning is the chief cause of a low specific gravity in an unadulterated cement. This is due to the fact that freshly ground cement when exposed to the air rapidly absorbs moisture and carbon dioxide. A month's seasoning will often reduce the specific gravity from 3.15 to 3.08, or thereabouts, and a long period of seasoning may reduce it to 3.00.\* Drying seasoned samples at 212° F. will slightly raise the specific gravity while igniting will, in general, raise the specific gravity to the original value. Seasoning the clinker lowers specific gravity.

The chemical composition of a cement also affects its specific gravity. Cements with high contents of iron oxide will have specific gravities 0.05 to 0.10 higher than those with low iron contents, provided both have been subjected to similar storage conditions.

Formerly the degree of calcination was supposed to affect the specific gravity, but numerous experiments have completely disproved this theory.† The effects of fineness of grinding upon specific gravity are slight. Very finely ground cements on account of the readiness with which they absorb moisture and carbon dioxide are likely to have lower specific gravities than cements made from the same materials but more coarsely ground.

#### RESULTS OF VARIOUS TESTS ON PORTLAND CEMENT

**355. General.**—We shall now consider some results of tests on Portland cement pastes and standard sand mortars. Only those results which are especially affected by the cement itself will receive attention herein. The effects produced upon mortars by variables in the aggregate will be considered in Ch. XIV.

**356. Strength Tests.**—From an elaborate series of tests reported in *Bulletin* No. 331, of the United States Geological Survey, the strength-age curves shown in Figs. 13, 14 and 15, have been compiled. The brands of cement used in these experiments were Alpha, Atlas (western), Star, Iola, Lehigh, Medusa and Vulcanite. The method adopted in making the tests was essentially the same as that advocated by the A.S.T.M. Each plotted point in the figures represents the average of 27 to 30 tests excepting in the diagram of the transverse tests, in which each point represents the averages of 150 tests. Table 3 contains the average results

\* For example see *Chem. Engr.*, Vol. 6, p. 19; or Taylor's *Practical Cement Testing*, p. 48.

† See *Chem. Engr.*, Vol. 6, p. 17; and *Proc. Inst. of Civil Engr.*, Vol. 166, p. 342.

low seven-day strengths become any stronger at the end of one year than those possessing higher initial strengths; neither do the slopes of

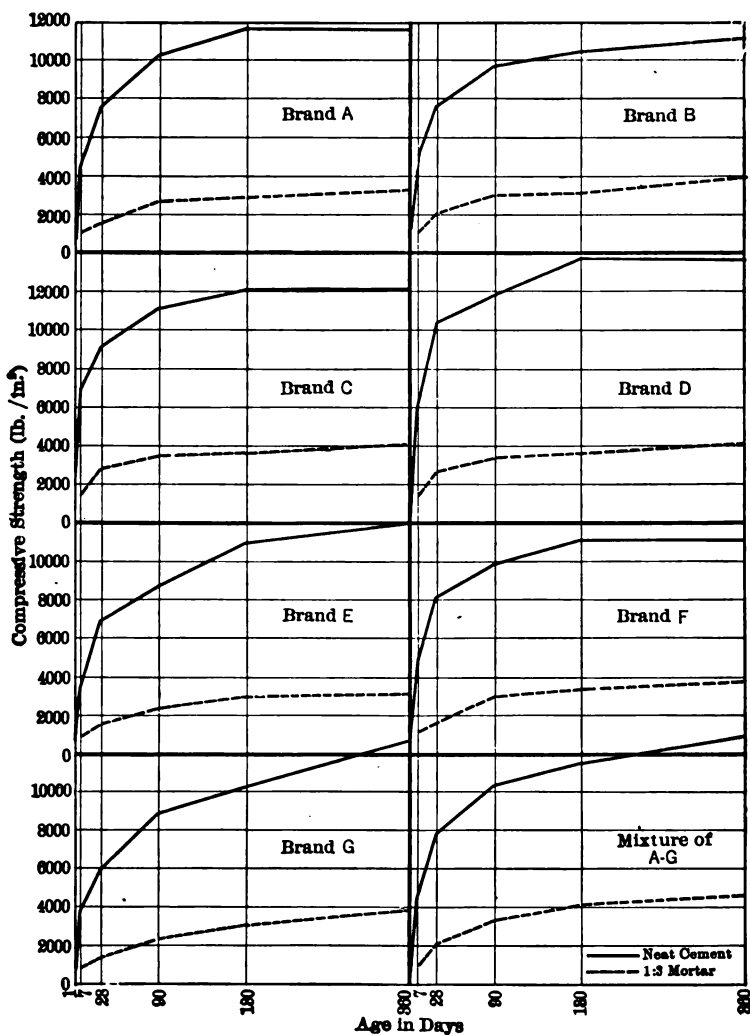


FIG. 14.—Variations in Compressive Strength with Age for Neat and 1 : 3 Standard Sand Mortar. (Specimens were 2-in. Cubes.)

the age-strength curve at the one-year period indicate that cements of low early strength will ultimately become the strongest.\*

\* The relative value of those two classes of cements and the advisability of specifying that the results of twenty-eight-day tests should show a percentage increase over the seven-day values has been the cause of much controversy. See *Engr. News*, Vol. 54, p. 63, 124, 149, 206.

TABLE 3.—AVERAGES OF CHEMICAL ANALYSES OF PORTLAND CEMENTS A-G AND MIXTURES

(See Figs. 13, 14, and 15)

Element.	PERCENTAGE IN CEMENT.							
	A	B	C	D	E	F	G	Mixture A-G
SiO <sub>2</sub> . . .	21.99	20.75	20.88	21.61	23.25	22.14	22.47	22.01
Al <sub>2</sub> O <sub>3</sub> . . .	7.24	7.72	7.91	7.88	5.32	7.32	6.94	6.78
Fe <sub>2</sub> O <sub>3</sub> . . .	3.39	2.59	2.69	3.18	3.27	3.02	2.79	3.21
CaO . . .	62.53	62.75	62.98	62.56	63.14	62.36	60.42	62.74
MgO . . .	2.37	2.61	2.85	2.37	3.01	1.61	3.23	2.64
SO <sub>3</sub> . . .	1.19	1.66	1.46	1.52	1.32	1.58	1.67	1.46
Undeter. . .	1.29	1.92	1.22	0.87	0.69	1.98	2.44	1.17

TABLE 4.—AVERAGES OF PHYSICAL PROPERTIES OF PORTLAND CEMENTS A-G AND MIXTURES

(See Figs. 13, 14, and 15)

	A	B	C	D	E	F	G	Mixture. A-G.		
Specific gravity.....	3.123	3.103	3.129	3.127	3.148	3.104	3.103	3.122		
Temp. (° F.) Water.....	75.0	74.7	74.5	74.4	74.4	74.1	74.1	69.6		
Temp. (° F.) Air.....	77.1	75.7	75.0	75.7	75.6	75.2	75.2	71.5		
Water (per cent).....	20.9	22.5	22.0	21.0	21.1	21.5	20.7	21.5		
Time of set (min.) {	Vicat {	initial.....	142	117	169	213	144	142	200	
		final.....	338	273	326	357	397	334	346	383
	Cilmore {	initial.....	215	195	249	244	261	210	222	248
		final.....	358	322	361	393	426	375	399	434
Fineness—% residue {	100	7.5	1.0	6.6	7.6	7.2	6.3	4.8	6.6	
	on sieve No. { 200	24.3	15.2	23.5	24.5	24.4	20.7	23.4	24.3	
* Soundness.....	2A $\frac{1}{4}$ "	{ 1A $\frac{1}{4}$ " 1A $\frac{1}{8}$ "	1A $\frac{1}{4}$ "	2A $\frac{1}{4}$ "	1A $\frac{1}{4}$ "	{ 3A $\frac{1}{4}$ " 1A $\frac{1}{4}$ "	O. K.	O. K.		

\* Nine or ten pats were made for each test on every cement. The legend indicates the number, kind of test and amount of warping; thus 2A  $\frac{1}{4}$  in. means two air pats had warped  $\frac{1}{4}$  in. at twenty-eight days.

Complete records of tests of cement extending over a long period of time are rare. In most cases the record of the properties of the cement, other than the strength, and the methods and conditions surrounding the curing of the specimens are very incomplete. Furthermore, the results of such tests are apt to be misinterpreted and a much broader significance given to them than is warranted by careful consideration of the facts. In general this may be stated of the strength of Portland cement specimens cured in clean fresh water. Usually the maximum tensile strength is attained within one year; thereafter it fluctuates, in most

cases showing some retrogression. Inasmuch as complete long-time compressive tests are still more rare than long-time tensile tests we can only say that, in general, the compressive strength continues to show a progressive increase for a longer period of time than does the tensile strength.

The curves in Fig. 16 show the results of long-time tests on six brands of Portland cement summarized in the Fourth Annual Report of the Metropolitan Water and Sewerage Board of Massachusetts. Fig. 17 illustrates the effects of curing in water and in moist air on the long-time strength of Portland cement mortars. These tests were reported by S. W. Hartwell, of the Vulcanite Portland Cement Company, in the *Engr. News*, Vol. 67, p. 846. Each point represents the average strength

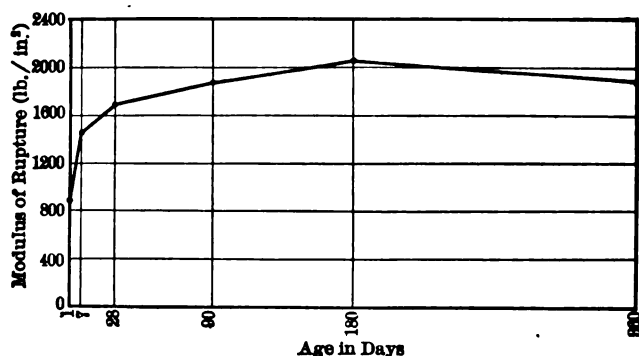


FIG. 15.—Variations in Transverse Strength with Age for Neat Cement Prisms Made of a Mixture of Brands A-G. (Specimens 13×1×1 in. were tested on a 12-in. span with center load.)

of five briquettes which were made and tested by the same operator. The sand used in the tests of Figs. 16 and 17 was crushed quartz \* which passed a No. 20 sieve and was held on a No. 30 sieve. For other long-time tests on tensile strength see Figs. 2 and 6, Ch. X.

In considering the results of long-time tensile tests, one should always bear in mind that the brittleness of cement test-pieces increases with age and that slight errors in the grips or bearing surfaces which produce an eccentric loading, will affect the strength of old specimens more seriously than the strength of young specimens.

**357. Expansion and Contraction Due to Variations in Moisture Content.**—Owing to colloidal nature, Portland cement pastes and mortars

\* This was the standard fine aggregate in general use in this country until 1904. It was more variable in granulometric composition than Ottawa standard sand. According to W. P. Taylor, 1 : 3 briquettes made of standard sand are from 20 to 30 per cent stronger at seven or twenty-eight days than those made of crushed quartz.

undergo a volumetric contraction when they harden in air and an expansion when hardening takes place under water. A few reliable experiments have been made to determine the magnitude of such action and the duration of time over which it lasts. In a paper by Prof. A. H. White, read before the Am. Soc. for Testing Materials (see *Proceedings*, Vol. 11, p. 531), there is given a summary of the main experiments upon this subject and an account of some important experiments made by White.

The cements used by White passed the standard soundness tests and standard methods of manipulation were employed in making the  $1 \times 1 \times$

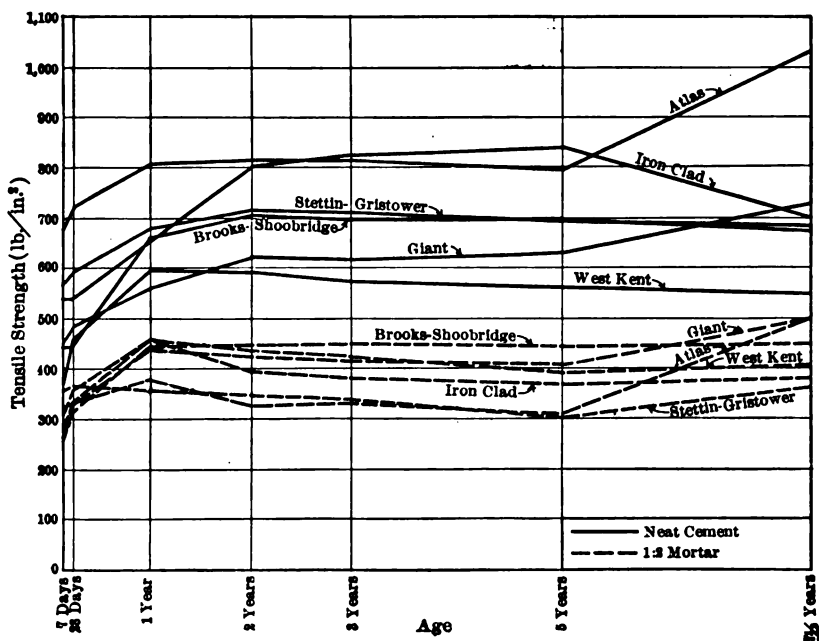


Fig. 16.—Tensile Strength-Age Curves for Six Brands of Portland Cement.

4-in. specimens. After curing in the moist closet for twenty-four hours an initial reading of the length of each prism was obtained by means of a special micrometer placed in contact with glass plates molded into the ends of the specimen. The probable error of the apparatus was about 0.003 per cent.

Fig. 18 shows the average contraction of air-cured neat cement at various ages up to four years. Five or six different brands are represented up to two years and three brands for the remainder of the time period. The effect of prolonged seasoning on contraction was negligible.

Fig. 19 \* gives the expansion time curves for water-cured neat prisms

\* Taken from a later paper in *Proc. A.S.T.M.*, Vol. 14, p. 204.

made of four brands of cement and also shows the effects of removal from water and subsequent immersion. It will be noted that the bars shrank very slowly after removal from water but expanded very rapidly within

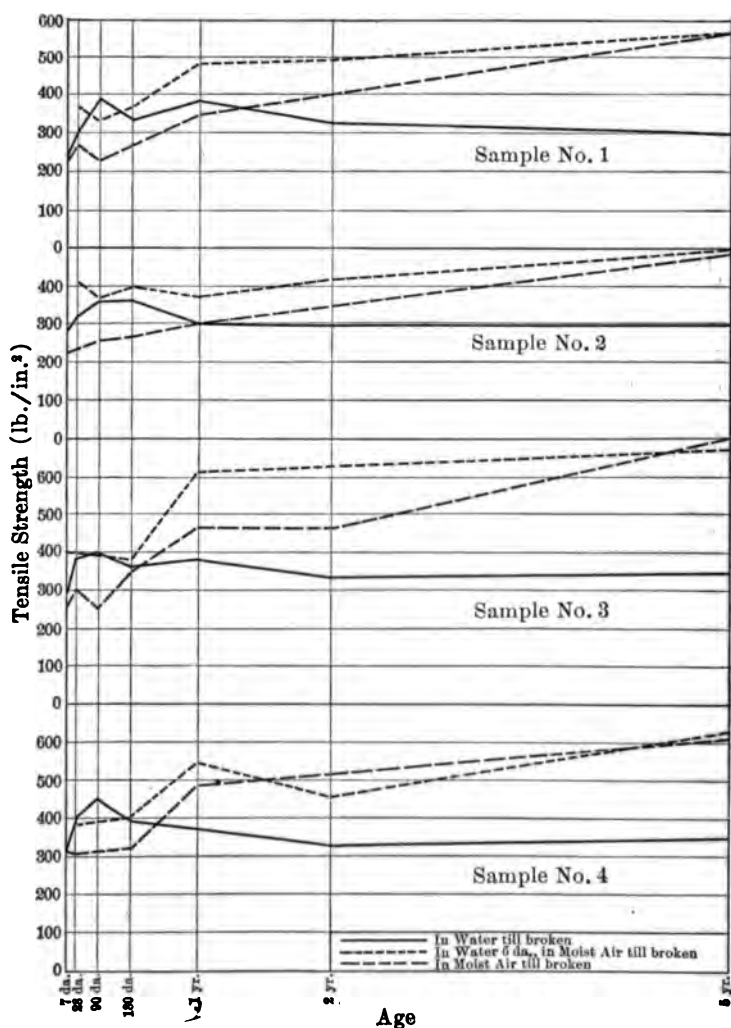


FIG. 17.—The Effect of Storage upon the Tensile Strength of 1 : 3 Crushed Quartz Mortars Made from Four Different Samples of Portland Cement. (Hartwell.)

a day when subsequently immersed. The behavior of the duplicate bars 146A3 and 146A4, well illustrate this phenomenon. The former, after being subjected to short immersion periods and long-drying periods, exhibited no pronounced change in mean length during three years treat-



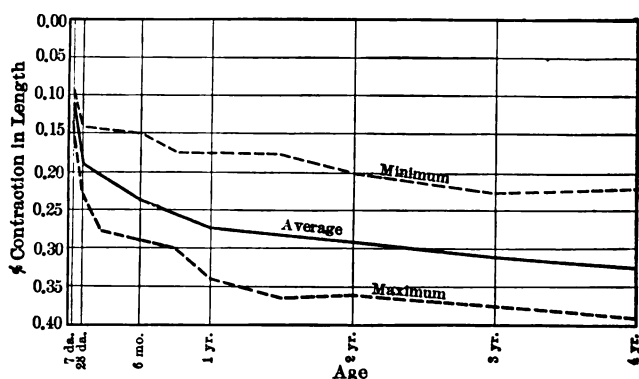


FIG. 18.—The Linear Contraction of Prisms of Neat Cement Stored in Air. (White.)

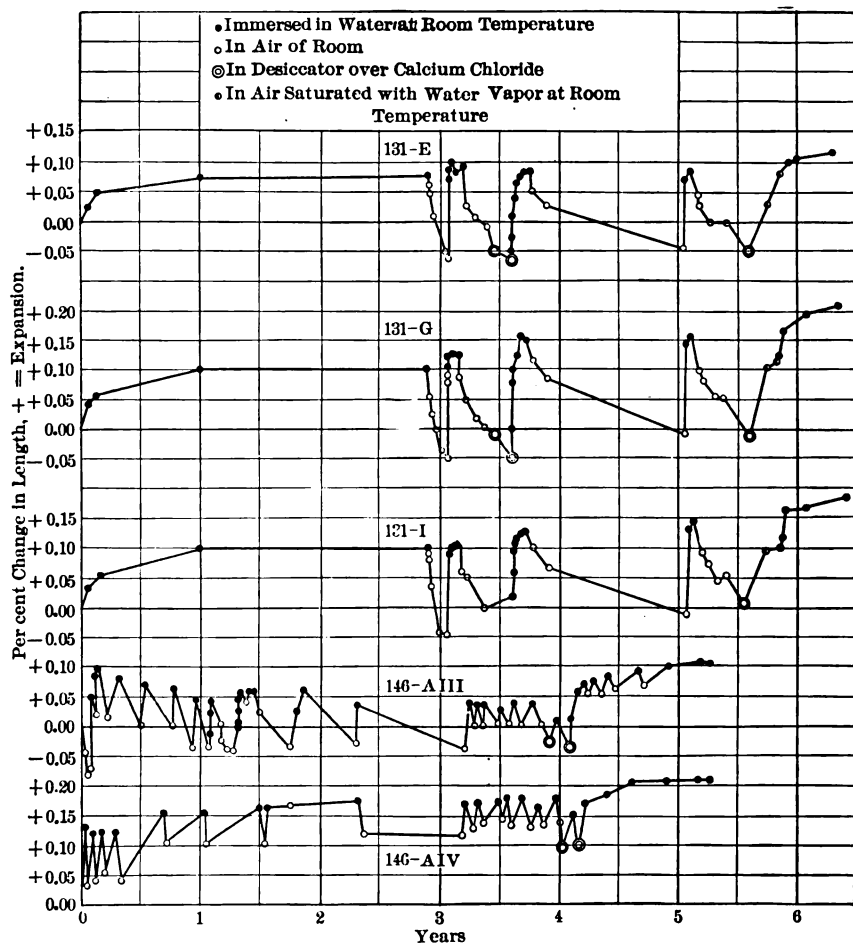


FIG. 19.—Changes in Length of Neat Cement Bars when Alternately Wet and Dried at Room Temperature. (White.)

ment, whereas bar 146A4, which received long periods of soaking and short-drying periods, increased in mean length about 0.15 per cent during the same time. No diminution in the proportionate expansion or contraction of the bars appeared after repeated applications of the alternate wetting and drying process. In several instances the changes in length increased with repetitions of this treatment. White ascribes the water attack on unhydrate cement grains to be the cause of this

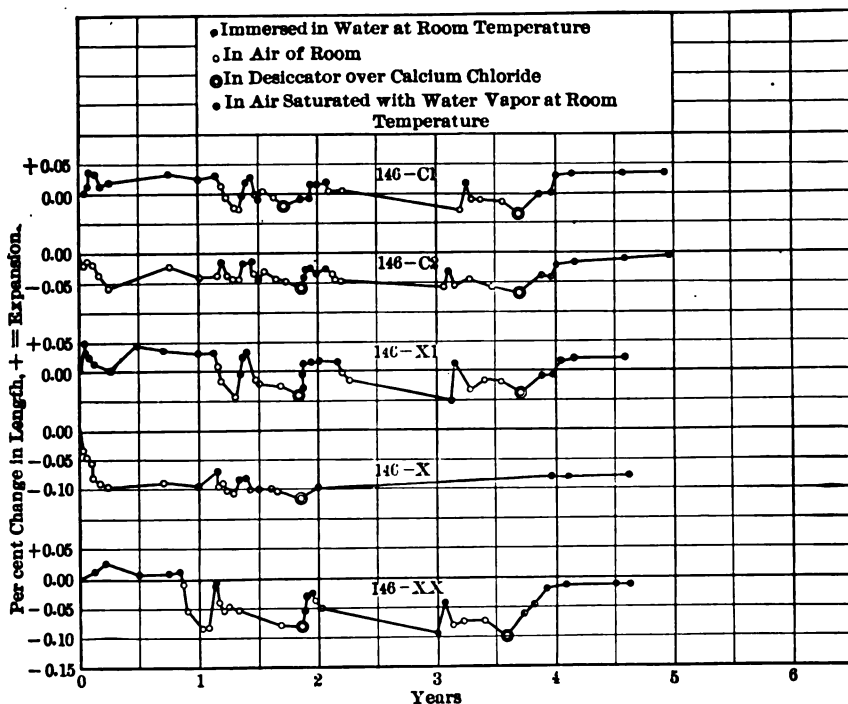


FIG. 20.—Changes in Length of 1 : 3 Standard Sand Mortar Bars when Alternately Wet and Dried at Room Temperature. (White.)

action. A saturated atmosphere caused expansion comparable to that obtained by immersion in water.

The expansion and contraction of 1 : 3 standard sand mortar prisms subjected to various periods of immersion and drying are shown in Fig. 20.\* It will be observed that the mortar bars attained the maximum changes more quickly than the neat specimens but the changes were much less. Alternate wetting and drying caused more rapid changes in the mortar specimens than in those made of neat cement. Prisms cut from sound and strong sidewalk tops, which had seen twenty years' service,

\* *Proc. A.S.T.M.*, Vol. 14, p. 204.

exhibited changes in length similar in kind and intermediate in magnitude to the changes observed in the neat and mortar bars. Experiments on compound bars made of equal layers of neat cement and 1 : 3 mortar led White to the conclusion that such specimens expanded and contracted together but not at the same rate nor to the same degree, the difference in expansion varying from 0 to 0.15 per cent.

Table 5 gives average values of expansions obtained by the U. S. Bureau of Standards Laboratory for neat and 1 : 3 mortar prisms made from ten different American brands of Portland cement.

TABLE 5.—AVERAGE LINEAR EXPANSION OF NEAT AND 1 : 3 STANDARD SAND MORTAR PRISMS OF TEN DIFFERENT BRANDS OF PORTLAND CEMENT CURED IN WATER

(*Proc. A.S.T.M.*, Vol. 15, p. 141.)

Mix.	PERCENTAGE LINEAR EXPANSION AT THIRTEEN WEEKS.*					
	NORMAL CEMENT.			REGROUND 87-93% <No. 200 MESH.		
	Maximum.	Minimum.	Average.	Maximum.	Minimum.	Average.
Neat.....	0.120	0.057	0.066	0.132	0.056	0.085
1 : 3.....	0.024	0.005	0.012	0.027	0.005	0.013

\* Average results represent twenty specimens. Each specimen was measured several times with a Berry strain gage.

The above results are in accord with those gotten from tests of small specimens by other investigators. Too much importance, however, should not be attached to the numerical values until experiments covering a wider range in variables have been made. Evidently the effect of increasing the proportion of sand is to diminish the changes in volume resulting from wetting and drying, so that volume changes in concrete are less than those indicated above; furthermore, the size of the specimen and the ratio of exposed surface to volume of specimen no doubt appreciably affect the rate and possibly the degree of such volumetric changes. The experiments indicate, however, that cement should be thoroughly mixed with sufficient water to insure complete hydration of all particles and hardened in a moist medium if volume changes, due to variations in moisture content, are to be minimized. For the results of experiments on concrete, see Art. 522.

**358. Effect of Remixing and Retempering on Strength of Cement.**—From the results of tests made at the Watertown Arsenal there has been plotted Fig. 21. This diagram shows the strengths of 6-in. neat cement cubes made from pastes which were remixed by hand after wetting for different intervals of time. The initial percentage of water is indicated

in the figure. Sufficient additional water was added whenever the paste became dry to secure the original consistency.

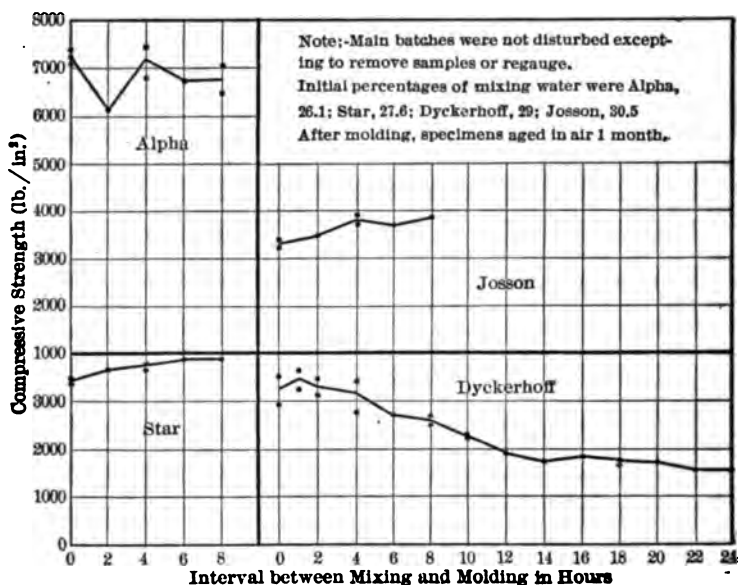


FIG. 21.—Effect of Remixing after Setting upon the Compressive Strength of Neat Cement Cubes.

A remarkable test on the effect of continuous agitation and retempering upon the strength of Star Portland cement is recorded in *Tests of*

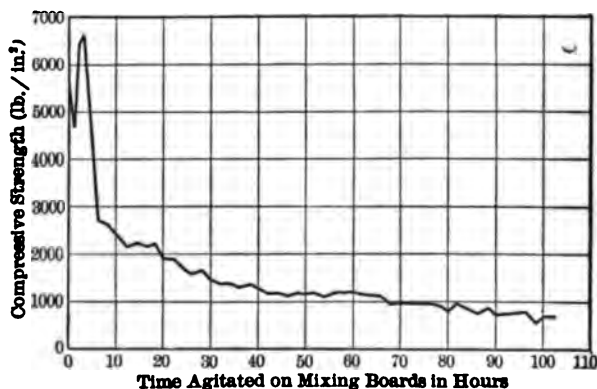


FIG. 22.—The Effect of Continuous Agitation during Setting on the Compressive Strength of Neat Cement Cubes. Age at test, one month.

*Metals*, 1901, p. 508. The main batch of neat cement paste was initially gauged with 32.9 per cent of water, retempered at intervals of one to four

hours to maintain initial consistency and was continuously agitated for 102 hours after mixing. Portions of the batch were separated from time to time and molded into 4-in. cubes. Fig. 22 shows the average strength of pairs of cubes plotted against the length of time which they were agitated.

Table 6 shows the strengths at one month of grouts made from various cements which were allowed to remain for fifteen or sixteen hours in the mixing board and were then remixed with an addition or removal of water, if necessary, to obtain the consistency ordinarily used by masons.

TABLE 6.—EFFECT OF RETEMPERING ON THE COMPRESSIVE STRENGTH OF NEAT CEMENT GROUTS \*

Specimens were 6-in. cubes. Age at test = 1 mo.

Brand.	Kind of Cement.	PER CENT WATER (BY WT.) AT		Interval between Mixing and Molding (Hr.).	No. of Spec.	COMPRESSIVE STRENGTH (L.B. PER SQ. IN.)		
		Mixing.	Molding.			Max.	Min.	Average.
Alpha.....	Portland	44.9	43.9	15	8	3706	3302	3480
Dyckerhoff..	Portland	43.3	43.3	15	8	2250	1908	2113
Josson.....	Portland	48.6	43.7	15	8	2304	1976	2087
Steel.....	Slag	50.9	50.9	15	8	585	519	554
Mankato....	Natural	68.8	68.8	16	7	316	255	294
Norton.....	Natural	59.0	64.5	15	8	377	305	343

\* Tests of Metals, 1901, p. 520.

**359. Effects of Low Temperatures on the Strength of Cement.**—In general it may be stated that the setting of cement proceeds at a very slow rate when the temperature falls below 40° F. When the temperature falls below freezing the particles of cement in unset specimens are separated by the expansion of water in freezing. A minimum amount of water should, therefore, be used in cement work subjected to freezing temperature in order that this expansive action may be as small as possible. Alterations in freezing and thawing before the cement has attained hard set generally cause a loss in cementing power owing to the repeated breaking of the bond between adjacent particles. If the work freezes before setting but thaws without refreezing it will in time secure full strength if plenty of moisture is provided for proper curing.

From the result of a very large number of compression tests on neat cement made at the Watertown Arsenal, the data in Table 7 have been selected. These tests show that there is considerable chemical activity in neat cement when setting at 0° F. It appears that neat cement specimens subjected to such low temperatures immediately after mixing gain strength at a very much slower rate than specimens cured at room tem-

peratures; but that after several years the test-pieces stored at low temperatures develop a considerable proportion of their normal strength. It also appears that specimens hardening at 70° F. for a given period will

TABLE 7.—THE EFFECTS OF LOW TEMPERATURES ON THE COMPRESSIVE STRENGTH OF 2-IN. CUBES OF NEAT PORTLAND CEMENT

(Tests of Metals 1901, 1902 and 1907.)

Brand of Cement.	Per Cent Water.	TIME OF SETTING IN AIR AT TEMPERATURES OF			Total Age. Days.	COMPRESSIVE STRENGTH * IN LB.-IN. <sup>2</sup> AFTER STORAGE.	
		70° F. Days.	0° F. Days.	70° F. Days.		Treatment Indicated.	In Air at 70° F. for Total Age.
Star .....	23.4	0	31	1	32	1350	4570
		0	31	7	38	2340	4820
		0	89	1	90	1720	
		0	89	30	119	3620	4410
		0	1 yr.	1	.....	2724	
Star .....	24.0	0	5 yr.	1	.....	3250	
Alsen .....	28.2	0	30	1	31	986	3900
		0	30	7	37	2440	3450
		0	90	1	91	1210	4040
		0	90	29	119	2520	3510
		0	1 yr.	1	.....	1580	
Alpha .....	25.0	.....	.....	1	1	.....	582
		1	7	1	9	2400	4990
		1	179	1	181	3670	5910
	25.0	1	5 yr.	1	.....	6320	
		1	5 yr.	36	.....	8100	
Alpha .....	23.0	.....	.....	7	7	.....	5550
		7	9	1	17	5160	5730
		7	188	1	196	5350	5940
		7	5 yr.	1	.....	7310	
		7	5 yr.	41	.....	7650	
Atlas .....	24.0	.....	.....	1	1	.....	689
		1	7	1	9	2140	4130
		1	181	1	183	2950	5410
		1	5 yr.	1	.....	4160	
		1	5 yr.	40	.....	6780	
Atlas .....	24.0	.....	.....	7	7	.....	3730
		7	7	1	15	4210	4890
		7	18	1	192	4500	5790
		7	5 yr.	1	.....	6640	
		7	5 yr.	40	.....	7410	

\* Each result is averaged from five or more tests.

attain a greater strength than specimens which are allowed to harden for a like period after an exposure to freezing temperatures. If neat specimens are allowed to set for one day at room temperature before being subjected to freezing temperatures, the rate of growth in strength is more rapid than if immediately frozen.

In a paper before the Am. Soc. of C. E.\* Matthews and Watson described experiments on the effect of frost on cement and cement mortar. From their results, it appears that after setting twenty-four hours, at normal temperatures (60° F.), a light frost (29° F.) temporarily lowered the strength of briquettes, but at the end of one month specimens cured in this temperature were equal in strength to those normally treated. Specimens frozen immediately after mixing exhibited only two-thirds of the normal strength at the end of either the seven or twenty-eight day period. A heavy frost, temperature 15° F., seriously affected the strength of neat and 1 : 3 mortar briquettes at both of the above periods even though the briquettes had set for twenty-four hours before being subjected to freezing. Mortar briquettes were more speedily weakened by light frost, but they recovered their strength more rapidly than the neat specimens. These experimenters also found that the percentage of water chemically combined with the cement was considerably less for specimens hardening in freezing temperatures than for those normally cured.

Gilman and Osterbind reported † a series of tests on neat briquettes which indicated that the tensile strength was most seriously impaired by alternate freezing and thawing before setting had taken place, provided such treatment occurred in air and was followed by a storage in air at 70° F. When briquettes were stored in water and subjected to alternate freezing and thawing followed by air-curing the decrease in strength, although equal to 50 per cent of the strength of specimens water-cured at normal temperature, was not so great as under above conditions.

**360. Effect of High Temperatures on the Strength of Neat Portland Cement.**—In *Tests of Metals*, 1902, a report is made of a number of tests on 4-in. cubes of neat cement which were cured for one year in air or water and then gradually heated to temperatures of 600° to 1000° F. The cubes were cooled in asbestos or sawdust and aged for four days to four months before they were tested. After heating most of the cubes showed faint cracks which gradually enlarged when the cubes were allowed to stand in air for several days. In several of the specimens subjected to temperatures of 900° F. these cracks became so large that the specimens were rendered unfit for testing. Specimens subjected to temperatures of 800° F. or above showed a marked decrease in strength,

\* *Trans. A.S.C.E.*, Vol. 64, p. 320.

† *Engr. Rec.*, Vol. 51, p. 388.

especially those hardened in water. Of the cubes made from Dyckerhoff cement and gauged with 29 per cent of water, those hardened in air showed a loss of weight varying from 7.7 per cent after being subjected to 700° F. to 10.5 per cent after 1000° F.; those hardened in water lost 17.7 per cent after being heated to 1000° F.

**361. Experiments on the Rise in Temperature During Setting.**—Considerable attention has been devoted to showing a relation between the rise in temperature during setting and the time of set of cement. No such relation has ever been established and it is probable that none exists. However, the possibility that the quantity of heat generated in

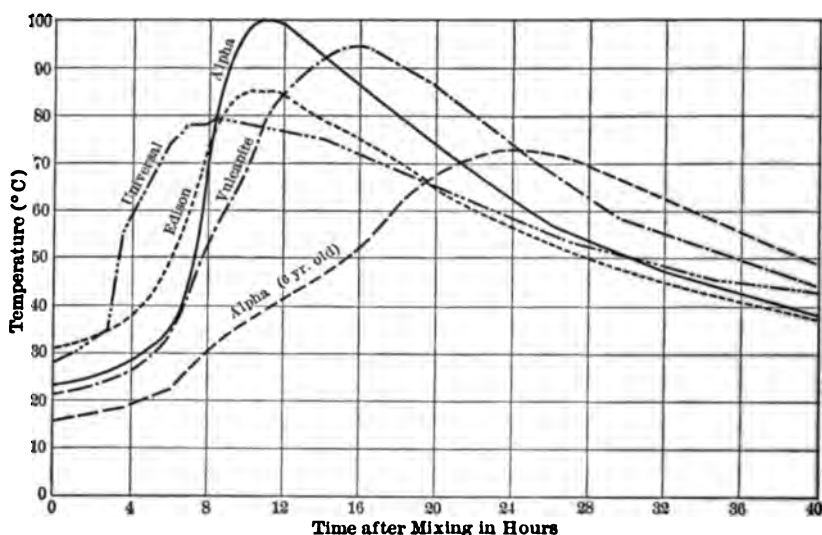


FIG. 23.—Temperatures Acquired by 12-inch Cubes of Neat Portland Cement while Setting in Plank Molds in a Room of Normal Temperature. (*Tests of Metals*, 1907, p. 129.)

setting may be an index of the quality of different samples of the same brand of cement is worth consideration.\* Furthermore, since the heat generated in setting doubtless affects the resistance offered to freezing, the typical temperature-time records for setting cement pastes presented in Fig. 23 are of interest. The pastes from which these cubes were made were gauged with 25 per cent of water. With the exceptions noted in the figure, all cements were ground shortly before the tests were made.

**362. The Resistance of Neat Cement to the Action of Alkali Waters and Sea Water.**—Under laboratory conditions neat cement may be disintegrated by the combined chemical and mechanical action of waters

\* See paper by L. N. Beals, Jr., *Proc. A.S.T.M.*, Vol. 13, p. 720.



containing various salts such as, the sulphates of magnesia and sodium, the chlorides of magnesia, sodium and calcium, and the carbonate of soda. The sulphates and chlorides are chemically active in removing lime from the cement, while the carbonate of soda alone or in solution with sodium sulphate or sodium chloride withdraws silica.\* If the test-pieces are subjected to alternate wetting and drying a mechanical action greatly accelerates the breakdown of the cement. Under such conditions, crys-

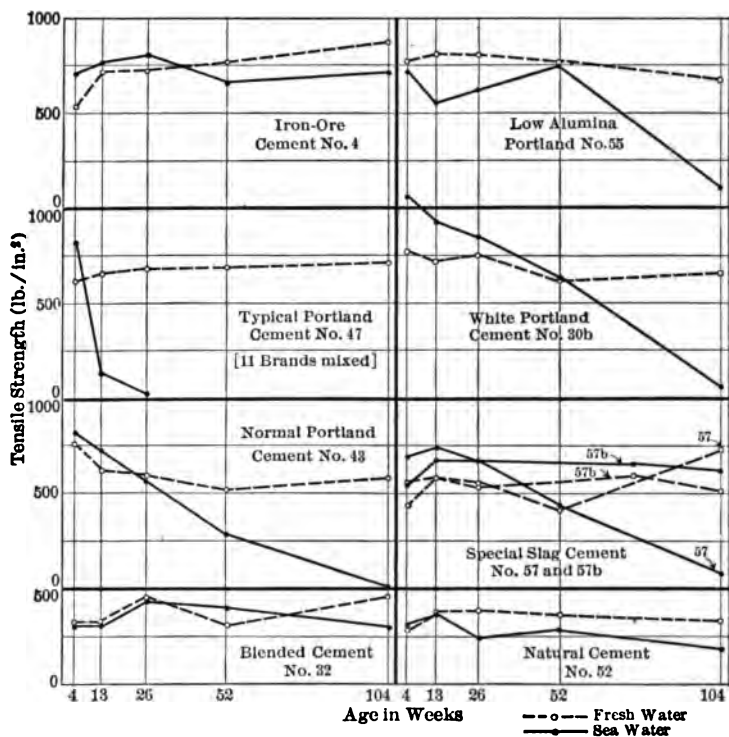


FIG. 24.—Neat Tensile Strengths of Different Cements Stored in Fresh Water and in Sea Water after One Day in a Moist Closet. (*Technologic Paper No. 12*, U. S. Bureau of Standards.)

tals of large size are rapidly formed and expansive forces are produced. Under the action of these forces neat cement pastes are disintegrated more rapidly than lean mortars.†

\* *Action of the Salts in Alkali Water and Sea Water on Cements*, *Technologic Paper*, No. 12, U. S. Bureau of Standards, by Bates, Phillips and Wig. (Excellent bibliography.)

† *The Effect of Alkali Water on Cement Mortars*, A. J. Fisk, *Eng. News*, Aug. 18, 1910.

Portland cements of high iron content and some of the special slag and tufa cements, in which the silica content is high, are thought by many to possess great resistance to the action of sea water.\* In view of tests made at the Bureau of Standards,† however, there appears to be no relation between chemical composition of a cement and its resistance to the chemical attack of sea water. In proportions up to 2.5 per cent sulphur trioxide has no serious effect upon the resistance of the cement to sea water.‡

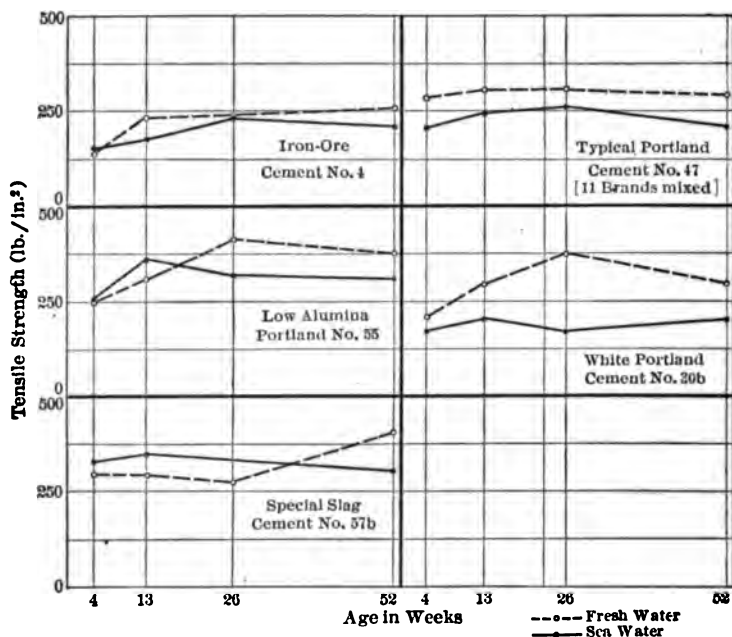


FIG. 25.—Tensile Strengths of 1 : 3 Standard Sand Mortar Briquettes, Made from Different Cements, Stored in Fresh Water and in Sea Water after One Day in a Moist Closet. (*Technologic Paper No. 12.*)

Magnesia combined with the cement is inert, but that contained in sea water is precipitated from a sulphate or chloride solution due to the greater affinity of the lime in the cement for the sulphur or chlorine radical.

In construction, however, due probably to the carbonizing of the lime, thus rendering it insoluble in sea water, and the formation of protective surface coatings, the destructive action is much less rapid. Concerning the mechanical disruptive action, it must be borne in mind that

\* *Action of Sea Water upon Hydraulic Cements* by Michaelis, *Trans. Inst. C.E.*, Vol. 107, p. 375; also *Engineering*, Vol. 63, pp. 457, 496, and 559.

† *Action of the Salts in Alkali Water and Sea Water on Cements*, *Technologic Paper*, No. 12, U. S. Bureau of Standards, by Bates, Phillips and Wig. (Excellent bibliography.)

‡ Report of German Portland Cement Mfg. Asso., *Cement Age*, Oct., 1911.

other porous materials like stone and brick are subject to similar action unless protected by an impervious surface.

It, therefore, seems advisable before condemning the use of cement in structures subjected to the action of alkali and sea waters to study methods of rendering the surface impervious. In such studies there should be employed specimens of large mass cured and treated under field conditions. The results of small specimen tests of neat cement are of practically no quantitative value in this connection, however much information they may yield concerning the nature of the action which takes place. Tests of briquettes of different cements cured in fresh water and in salt water are given in Figs. 24 and 25. Chemical analysis of the cements can be found in Table 8.

TABLE 8.—CHEMICAL ANALYSES OF CEMENTS

(See Figs. 24 and 25)

No.	4	55	47	43	30b	57	57b	32	52
Kind.	Iron * Ore Cement.	Low Alumina Port- land.	Mixed Brands, Port- land.	Normal Port- land.	White Port- land.	Special Slag.	Special Slag.	Blended.	Natural.
SiO <sub>2</sub> . . . .	23.44	20.37	21.50	22.07	22.66	27.77	29.48	24.00	22.53
Al <sub>2</sub> O <sub>3</sub> . . . .	2.98	3.64	8.12	6.95	8.61	13.87	15.37	9.69	8.98
Fe <sub>2</sub> O <sub>3</sub> . . . .	7.48	8.97	2.28	2.31	0.55	0.60	0.64	2.53	2.45
CaO . . . . .	61.86	61.42	62.23	62.33	62.46	44.09	42.23	49.15	45.75
MgO . . . . .	0.50	0.82	3.24	2.28	1.10	4.49	4.45	2.53	2.92
SO <sub>3</sub> . . . . .	1.72	1.19	1.45	1.54	1.64	1.22	1.83	1.67	1.81
Na <sub>2</sub> O . . . . .	0.20	1.54	0.18	0.31	0.40	0.42	0.56	0.71	0.39
K <sub>2</sub> O . . . . .	0.28	0.24	0.38	0.57	0.53	0.11	0.57	1.42	1.20
CO <sub>2</sub> . . . . .	0.69	1.01	0.12	0.45	0.63	2.10	0.64	4.43	8.60
CaS . . . . .	.....	.....	.....	.....	.....	2.84	2.25		
H <sub>2</sub> O . . . . .	0.42	.....	0.38	0.45	0.36	.....	.....	0.45	
Ig. loss . . .	0.09	1.06	0.39	0.92	1.07	2.68	2.16	3.47	3.53
Total . . . .	100.17*	100.26	100.27	100.16	100.01	100.19	100.18	100.05	100.16

\* Contains 0.51 % CuO.

**363. Effects of Oils on Neat Cement.**—In general it may be stated that most mineral oils produce little if any effect upon hard-set specimens of neat Portland cement.\* When incorporated into the mixing water they retard the set and decrease the strength. Animal and vegetable oils, which contain acid fats, will attack the lime compounds in neat cement and form lime soap.

Toch † states that this disintegration may be the result of expansion

\* J. C. Hain, *Engr. News*, Vol. 53, p. 279.

† Ibid., 419.

during the crystallization of oleate and stearate of calcium. Such action is more pronounced in neat cement than in mortar or concrete specimens. It is, therefore, imperative that care should be exercised to use only small quantities of mineral oils for lubricating molds and never employ vegetable or animal oils for such purposes.

**364. Effects of Sugar on Cement.**—The addition of as small an amount as one-eighth of one per cent of sugar to cement has been reported to produce a marked delay in the time of set and practically to destroy the early strength.\* At an age of two or three months additions of sugar, less than 2 per cent, appear to increase the strength. By some, the action of sugar is attributed to the formation of a soluble calcium saccharate ( $C_{12}H_{22}O_{11} \cdot CaO + 2H_2O$ ).

\* Parsons, *Engr. News*, 1887, Vol. 2, p. 461; also see series of letters in Vol. 69, p. 126, 127, 478, 1077.

## CHAPTER X

### NATURAL AND OTHER HYDRAULIC CEMENTS

#### NATURAL CEMENT

**365. Definition.**—Natural cement is made by burning a natural argillaceous limestone at a low red heat ( $1000^{\circ}$  to  $1300^{\circ}$  C.), which is sufficient to drive off carbonic oxide ( $\text{CO}_2$ ). The clinker will not slake to any extent and must be finely ground before it exhibits hydraulic properties.

**366. Process of Manufacture.**—The limestone, containing from 15 to 35 per cent clay, is burnt in vertical kilns 30 to 40 ft. high and 10 to 15 ft. in diameter. The common type of kiln consists of a cylindrical steel shell open at the top and lined with fire-brick. In operating a kiln, thick layers of limestone and thin layers of soft coal are alternately dumped into the top of the furnace and the burnt clinker is drawn off at frequent intervals from the bottom. As the limestone descends in the kiln, water is first driven off from the rock. At a temperature of about  $700^{\circ}$  C. magnesian carbonate begins to decompose. Lime carbonate dissociates at  $900^{\circ}$  C. and clay at a somewhat higher temperature. The alumina and iron oxide set free by the decomposition of the clay combine with the lime and magnesia and, if the final temperature be high enough, lime and magnesian silicates will be formed. The process is run continuously and about one-third of the charge, in the form of clinker, is daily withdrawn from the kiln.

On account of the variations in the quality of the raw material and on account of non-uniformity in burning different parts of the charge, a considerable portion of the resultant clinker is either under-burned or over-burned. According to Eckel from 10 to 33 per cent of the resultant product cannot be used. After the clinker has been removed from the kiln it is allowed to season in the air in order that any under-burned clinker may be slacked before grinding. Sometimes slacking is accelerated by steaming the clinker.

The burnt clinker is first passed through a stone crusher and then fed to some form of apparatus for grinding it to the requisite fineness. Formerly, all mills used the millstone grinders commonly employed in flour mills. More recently, however, a decided improvement in the fineness

of grinding has been effected by the introduction of ball mills, tube mills and other modern equipment used in grinding Portland cement.

Natural cement is sold either in barrels of 280-lb. capacity or in 90-lb. bags.

**367. Characteristics of Natural Cement.**—Natural cement is an impalpable powder varying in color from yellow to brown and in specific gravity from 2.80 to 3.00. It resembles hydraulic lime inasmuch as it is made from a natural argillaceous limestone and will set when mixed with water either in air or under water. On the other hand, natural cement clinker slakes but little, if any, when water is poured upon it.

Most natural cements are not so finely ground as Portland cements although much improvement in this important property has been effected in recent years.

Natural cement sets much more rapidly but with a less evolution of heat than Portland cement. For pastes of normal consistency the time

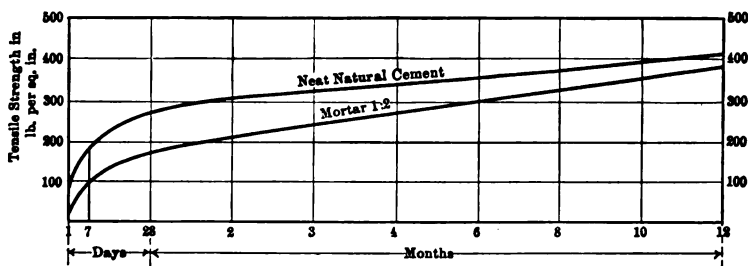


FIG. 1.—Tensile Strength-Age Curves for Neat and 1 : 2 Natural Cement Mortars  
(From Taylor's *Practical Cement Testing*.)

of initial set will usually vary from fifteen minutes to one hour and final set will generally occur within three hours. An excess of water greatly retards the setting of natural cement pastes. Aeration also retards the set of natural cement.

On account of the variability of the raw materials employed in manufacture, natural cements differ considerably in chemical composition. Even in the same brand considerable differences in the composition are common owing to variations in the rock and degree of calcination. The approximate ranges in amounts of the chief chemical compounds found in natural cements, as obtained from over 100 analyses given in Eckel's *Cements, Limes and Plasters*, Ch. XIX, are as follows: 30 to 60 per cent of lime ( $\text{CaO}$ ); 15 to 35 per cent of silica ( $\text{SiO}_2$ ); 1 to 25 per cent of magnesia ( $\text{MgO}$ ); 2 to 20 per cent of alumina ( $\text{Al}_2\text{O}_3$ ); 1 to 19 per cent of iron oxide ( $\text{Fe}_2\text{O}_3$ ); and, in general, less than 10 per cent of water ( $\text{H}_2\text{O}$ ), carbon dioxide ( $\text{CO}_2$ ), the alkalies ( $\text{K}_2\text{O}$ ,  $\text{Na}_2\text{O}$ ), and sulphur trioxide ( $\text{SO}_3$ ).

**368. Properties of Natural Cement.**—Because of variations in composition and manufacture, the properties of natural cements, even those of the same brand, often differ considerably. In Fig. 1 are shown the average tensile strength results of numerous tests made by W. P. Taylor on different brands of natural cement. Attention is called to the regular increase in tensile strength which accompanies an increase in the age of briquettes. Although natural cement gains its full strength much less rapidly than Portland cement, it does not, in general, exhibit marked

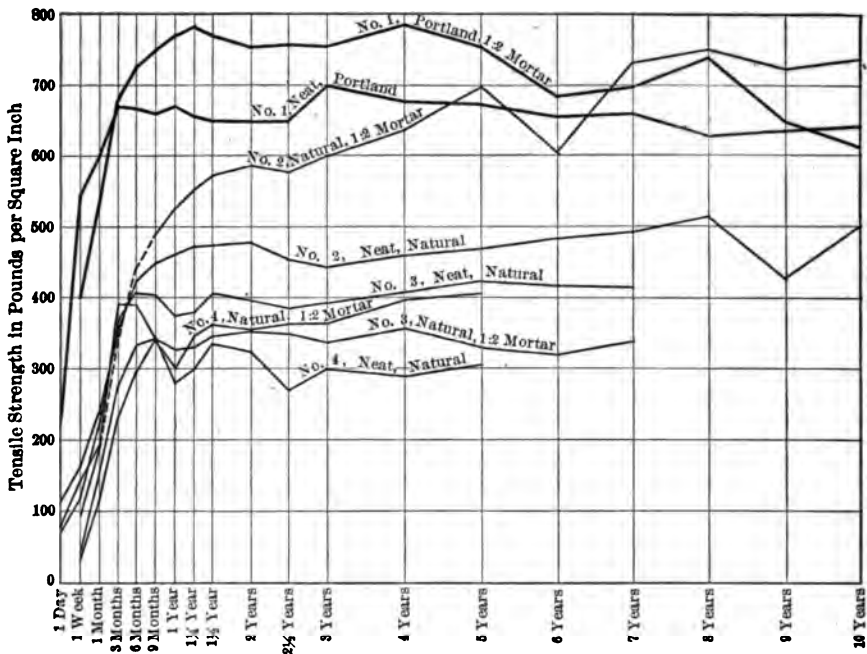


FIG. 2.—The Effects of Age on the Tensile Strengths of Natural and Portland Cements Used in the New Croton Dam. (G. G. Honness, *Trans. A.S.C.E.*, Vol. 76, p. 1038.)

retrogression in strength, even when tested neat. This fact is well illustrated by Fig. 2 which shows the results of long-time tensile tests on three brands of natural and one brand of Portland cement used on the New Croton Dam. In these tests the mortars were made with crushed quartz passing a No. 20 sieve and retained on a No. 30 sieve. The number of specimens per point on the diagram ranged from 15 to 14,740. Nearly all of the tests were made by one operator. In Fig. 3, the reduction in strength of natural cement mortars for increasing proportions of sand is indicated.

The results of compressive tests at the Watertown Arsenal on 4-in.

neat cubes made of four different brands of natural cement indicate the following ranges in strength for corresponding ages of specimens.

Age when broken.....	7 day	1 mo.	3 mo.	1 yr.	3 yr.	5½ yr.
Compressive strength (lb.-in. <sup>2</sup> ) Min.....	356	840	1110	1040	1320	1290
Max.....	566	1090	1530	1590	1866	2440

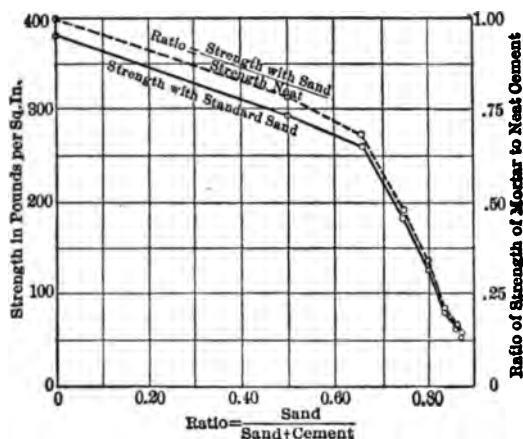


FIG. 3.—Showing Reduction of Strength of Natural Cement Mortar Six Months Old for Increasing Proportions of Sand. (Wheeler, *Rept. Chf. Engr.*, 1895, p. 2982.)

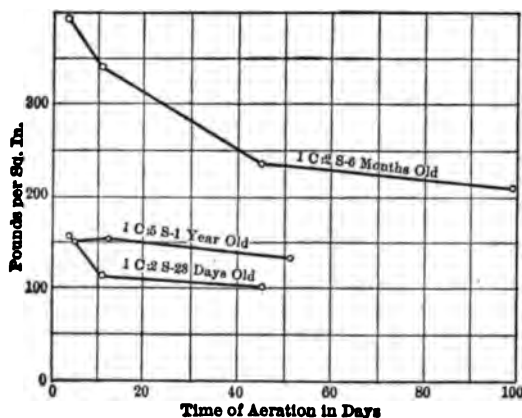


FIG. 4.—Effect of Aeration on the Tensile Strengths of Natural Cement Mortars. (Wheeler, *Rept. Chf. Engr.*, 1895, p. 2962.)

The specimens were made from pastes of normal consistency and were hardened in air. Nearly all of them were badly cracked at the end of the five and one-half-year interval.\*

Fig. 4 shows the effects of aerating the cement upon the strength of briquettes. The results indicate that natural cement should not be exposed to air over one month.

Regauging adversely affects the strength of neat natural cement mortars as Fig. 5 plainly shows. Simi-

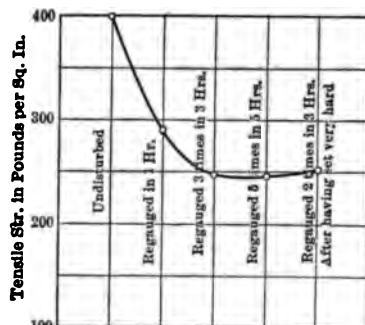


FIG. 5.—Effect of Regauging on the Tensile Strength of a Quick-setting Neat Natural Cement Mortar. Test age was six months. (*Rept. Chf. Engr.*, 1895, p. 2980.)

\* *Tests of Metals*, 1902, p. 369; 1904, p. 341; 1907, p. 133.



lar effects are reported in *Tests of Metals*, 1901, p. 522. In the latter, are also reported results of tests on neat natural and neat Portland cement grouts (see Art. 358), which were gauged with water and allowed to set overnight before retempering. The compressive strengths of these natural cement grouts after hardening in air for one month varied in general between 200 and 300 lb. per square inch. Regauged Portland cement grouts, similarly treated, exhibited about ten times this strength.

When subjected to low temperatures neat natural cement pastes are affected like Portland cement pastes but their strengths are very much lower.\* Alternate freezing and thawing, however, is more damaging to natural cement mortars than to those made of Portland cement. Consequently natural cement is not so well adapted as Portland cement for constructions which must be carried on in freezing weather.

Results of the effects of sea water on the strength of briquettes made from a brand of natural cement (52) appear in Fig. 24, Ch. IX.

**369. Uses and Production.**—Natural cement has been extensively used in sewer work, in masonry construction, and in monolithic or massive construction in concrete in which great strength was not required. Since the year 1899, however, the production of natural cement in the U. S. has steadily declined. This decline has been brought about largely by the decrease in the cost of Portland cement. In 1914 the production of natural cement was 751,285 bbl. valued at 46.8 cents per barrel. Only 12 plants reported any production in 1914.

### MISCELLANEOUS CEMENTS

**370. White Portland Cement.**—On account of the demand for a stainless white cement of high strength there have recently been placed upon the market several pure white Portland cements. These are employed in ornamental work both interior and exterior and in making building blocks, stucco, stainless mortar, etc. In order to obtain the pure white color it is necessary to use raw materials with a very low content of iron oxide. Generally less than one per cent of this oxide is present in the finished cement. Furthermore, it is necessary to use fuel free of pyrite and to burn at a temperature above the normal. Consequently white Portland cement is somewhat more expensive than the ordinary variety.

In physical properties, white Portland cements generally conform to the Standard Specification for Portland Cements, although the strength at one and seven days is sometimes low. Tensile strength-time curves for neat and 1 : 3 standard sand mortar briquettes made from a white

\* See Art. 359 and *Tests of Metals*, 1901, p. 540-582.

Portland cement (30b) are shown in Figs. 24 and 25, Ch. IX. The chemical analysis of this brand is given in Table 8, Ch. IX.\*

**371. Cements with High Iron Content.**—Because of the belief that a high alumina content is undesirable in cements for marine constructions, cements have been developed in which the alumina content has been largely replaced by iron oxide. Two grades of these cements have been marketed by German manufacturers,† the Iron-Portland cements, which are made by grinding a mixture containing 70 per cent or more Portland cement clinker and slag, and iron-ore cement, which is a true Portland made from raw materials having a high iron and low alumina content. In this country the latter type of cement has been made from the New Jersey greensand, a ferrous silicate, and shell-marl.‡ These cements do not ordinarily contain over 2 or 3 per cent of alumina. They have a somewhat higher specific gravity, are slower setting and somewhat weaker at one and seven days than the normal Portlands.

Whether such cements do offer superior resistance to the attack of sea water is a question on which authorities differ. The results of briquette tests on iron-ore cement (4), and a low alumina cement (55), which were cured in fresh water and in salt water, may be seen in Figs. 24 and 25, Ch. IX. Table 8, Ch. IX, shows the chemical analyses of these cements. From the results of these experiments and those of Candlot, Mr. P. H. Bates, who reports the tests, suggests that a very high silica content is probably of more value than the presence of an unusual proportion of iron oxide.

**372. Blended Cements.**—In 1893 a patent was taken out by T. L. Smidth for the manufacture of a cement which was made by grinding together silicious rock or sand and Portland cement. Although this cement was successfully employed in a number of constructions, it did not remain long on the market owing to the rapid decrease in price and increase in production of Portland cement. Recently, the U. S. Reclamation Service has made use of sand cement on the Arrowrock and Elephant Butte Dam, and the City of Los Angeles has employed tufa cement, a similar blended product, in the construction of a very large aqueduct. In these constructions the cost of Portland cement was made very high on account of the long haul so that the use of a blended cement effected a considerable economy. On the Reclamation projects this saving amounted to about 25 per cent of the cost of Portland cement. The cost of producing these blended cements, exclusive of the price paid for Portland, varied from 37 to 50 cents per barrel. In view of the use which

\* See also Properties of White Portland Cement by P. H. Bates, *Trans. Am. Ceramic Soc.*, Vol. 16, p. 551.

† Dr. W. Michaelis, Sr., has been a pioneer in this field.

‡ Eckel in *Eng. News*, Vol. 66, p. 157.

these cements have received, both in this country and abroad, a short description of them will be given.

**373. Sand Cements.**—Generally sand cements have been made by grinding together in a tube mill approximately equal volumes of pulverized silicious rock or sand and Portland cement. In the work of the Reclamation service it has been found desirable to use rocks in which there is a high content of soluble silica.\* Pure quartz is not a desirable element because it is insoluble, but basalts and sandstones often contain soluble silica.†

The reason for a soluble silica requirement is evident if Michaelis'

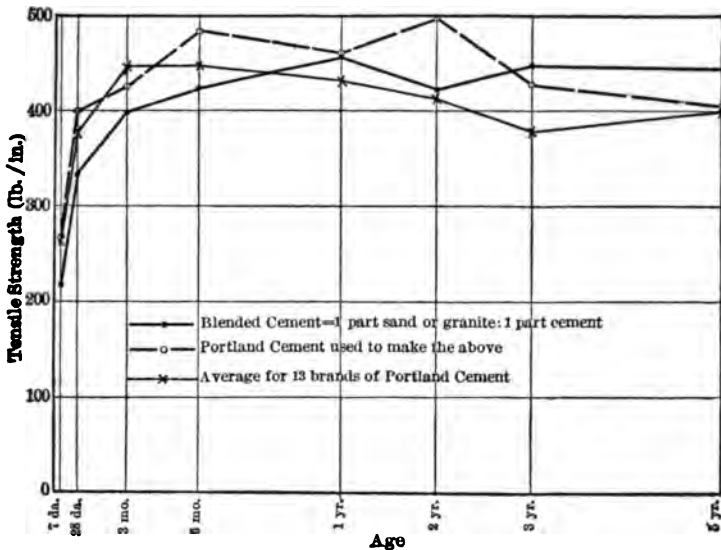


FIG. 6.—Tensile Strengths of 1 : 3 Mortar Briquettes of Standard Sand and Blended Cement Compared with Like Specimens Made of Portland Cement (Paul).

explanation of the setting of cement is accepted. From his theory it seems probable that the soluble silica in the rock reacts with the excess calcium hydroxide, which in a pure cement paste would be uncombined and in time crystallized, to form colloidal calcium hydro-silicate, which is the chief component in the hardening of the cement. It is very probable that the formation of this colloid is promoted by the reduction in the size of both cement and silica particles, which is effected by grinding them together. From Coghlan's experiments \* there appears to be produced no reduction in the percentage of voids by the addition of the fine rock.

\* R. R. Coghlan, *Engr. News*, Vol. 69, p. 1270.

† By soluble silica is meant hydrated silica which in a finely subdivided form is soluble in water.

are not so great. Mortars of puzzolan cements are tough but have little resistance to attrition. According to the report of the United States Army Engineers \* puzzolan cement mortars and concretes should not be used where they will be continually exposed to the air since such exposure produces disintegration by oxidation of the sulphides in the slag. Inasmuch as these cements possess hydraulic properties and are highly silicious, they are commonly believed to be less affected by sea water than Portland cements.

The tensile strength-time curves for briquettes made from a slag cement (57 and 57b), when cured in fresh water and in sea water are shown in Figs. 24 and 25, Ch. IX. The Chemical analysis of this cement is furnished by Table 8, Ch. IX.†

**378. Improved Cements.**—Some manufacturers of natural cement have improved their product by grinding with their natural cement clinker a small proportion of Portland cement clinker. By such adulteration the rate of hardening and the strength at early ages is increased. The compressive strength of 4-inch cubes of neat Bonneville improved cement is reported in *Tests of Metals*, 1902, 1904 and 1907. The cubes were made of normal consistency and set in air. Average values for five specimens follow:

Age.....	7 days	1 mo.	3 mos.	5½ yr.
Strength (lb./in. <sup>2</sup> ).....	620	1130	1560	2050 *

\* Average of three cubes. Specimens were cracked.

\* *Engr. News*, Vol. 46, p. 180.

† For further results see tests by W. K. Hatt, *Engr. News*, Vol. 45, p. 164.

## CHAPTER XI

### LIMES AND PLASTERS

#### LIMES

**379. Quicklime.**—Pure lime, generally called quicklime, is a white oxide of calcium. Much of the commercial quicklime, however, contains more or less magnesian oxide, which gives the product a brownish or grayish tinge. The specific gravity of pure lime is about 3.10. Essentially, the process of making lime consists in heating calcite ( $\text{CaCO}_3$ ), or magnesian limestone ( $x\text{CaCO}_3 + y\text{MgCO}_3$ ), to a temperature sufficiently high to drive off the carbon dioxide ( $\text{CO}_2$ ). For pure lime carbonate the temperature at which such dissociation takes place is approximately  $900^\circ \text{C}$ .

Since a considerable length of time is required to calcine limestone at such temperatures, it has been found good practice in operating kilns to use higher temperatures, depending upon the character of the impurities of the stone. However, to avoid burning, which seriously injures the setting properties, high magnesian limes should not be subjected to temperatures above  $1000^\circ \text{C}$ . and high-calcium limes should be burnt at temperatures lower than  $1300^\circ \text{C}$ .\*

The American Society for Testing Materials in 1915 adopted the following classification for quicklime.† (a) High-calcium; (b) calcium (c) magnesian; (d) high-magnesian. The society recognized two grades: *Selected*—a well-burned lime picked free from ashes, core, clinker, or other foreign material; and *Run-of-Kiln*—a well-burned lime, without selection. The requirements as to composition appear in the table on the next page:

**380. Burning of Lime.**—Limestone is usually burnt in some form of vertical kiln. The raw material is fed in at the top and the finished product drawn off through an opening in the side near the bottom. In general the stacks of these kilns consist of cylindrical steel shells lined with refractory brick. Kilns may be operated continuously or intermittently. To secure the greatest efficiency continuous operation is imperative. The common types of kiln are the mixed-feed and separate-feed kilns. In the mixed-feed type, bituminous coal and limestone are fed into the top of

\* *Trans. American Ceramic Society*, Vol. 13, p. 618, article on burning of limestones by A. V. Bleininger and W. E. Emley.

† See *Year Book of A.S.T.M.*, 1915, Serial Designation C5-15, for the specifications.

CHEMICAL COMPOSITION OF LIMES (A.S.T.M.)

Properties Considered.	HIGH CALCIUM.		CALCIUM		MAGNESIAN.		HIGH-MAGNESIAN.	
	Selected.	Run-of Kiln.	Selected.	Run-of Kiln.	Selected.	Run-of Kiln.	Selected.	Run-of Kiln.
Calcium Oxide, per cent. . . .	90 (min.)	90 (min.)	85-90	85-90	....	....	....	....
Magnesium Oxide, per cent. . . .	....	....	....	....	10-25	10-25	25 (min.)	25 (min.)
Calcium Oxide plus Magnesium Oxide, min., per cent.	90	85	90	85	90	85	90	85
Carbon Dioxide, max., per cent. . . . .	3	5	3	5	3	5	3	5
Silica plus Alumina plus Oxide of Iron, max., per cent. . . . .	5	7.5	5	7.5	5	7.5	5	7.5

kiln in alternate layers; in the separate-feed type, the limestone is not brought into contact with the fuel during the burning process. To accomplish this the fuel is burned in a grate which is attached to the sides of the kiln (see Fig. 1), and which is so arranged that the heat produced will ascend into the stack. The majority of the separate-feed kilns burn coal, some wood and a few producer gas. The mixed-feed kiln is more economical of fuel but does not produce as high-grade product as the separate-feed kiln. Most American concerns now use the separate-feed kiln to burn lime.



FIG. 1.—A Separate Feed Keystone Lime Kiln.

The rotary kiln which is widely used in manufacturing Portland cement has not met with favor in burning lime for two reasons. If the rotary kiln is used the rock must be finely crushed. This not only increases the cost of burning but also makes it necessary to grind or hydrate the burnt material.

In Germany continuous kilns of the ring type similar to those used in this country for burning clay products are much used. (See Art. 344.)

**381. Production Statistics.** — Lime is made in nearly every state and territory in the United States. The states which lead in production are: (1) Pennsylvania, (2) Ohio, (3) Maine, (4) New York.

The total output of the U. S. in 1914 was 3,380,928 short tons, valued at \$13,247,676. Nearly one-sixth of the lime produced is sold in hydrated state. About 40 per cent of the total production is used in construction. A considerable proportion of the remainder is utilized in making fertilizer, in chemical industries and in paper manufacture.

**382. The Slaking and Hardening of Lime.**—When 18 parts, by weight, of water is mixed with 56 parts of pure quicklime the mixture swells with an evolution of considerable heat, increases in volume about 300 per cent and forms calcium hydroxide  $\text{Ca}(\text{OH})_2$ . Limes high in magnesia, poor limes, will require more water and slake more slowly than high-calcium limes, fat limes. However, in order to form a paste easily worked under the trowel and to avoid burning, masons slake lime with a large excess of water. Porous limes or finely ground limes slake more rapidly than dense, lumpy limes. Underburning or overburning causes the lime to slake slowly and injures its strength. When used in construction lime paste is mixed with 1 to 4 parts of sand. By adding the sand, not only is the cost decreased but the shrinkage is also greatly reduced.

In the hydrating of lime, part of the calcium hydroxide crystallizes and part forms colloids. These components secure a certain amount of rigidity through evaporation and absorption of the surplus water by the surrounding masonry. Final hardening is attained, however, through desiccation of the water in the crystal or colloid mass and replacement by carbon dioxide from the air. Thus the paste is slowly converted into a carbonate of approximately the same composition as the original limestone. Obviously, owing to the inaccessibility of the interior of a masonry joint this hardening action progresses very slowly and, if the wall is quite thick, may take years for its completion.

When stored in air, quicklime soon decrepitates due to the absorption of moisture and carbon dioxide and is in time reduced to lime carbonate. Therefore, in order to preserve its hardening power, quicklime must be stored in barrels or other tight containers unless it is to be immediately used. Since experience has shown that ground lime keeps better than lump lime, manufacturers of ground lime can ship their product in open cars, although some prefer to use bags. When ground lime is thus exposed in shipment the outer layer of material air-slakes and thus protects the inner mass.

On account of the impurities, silica, alumina and iron oxide often present in limes, the amount of water and the time required for slaking vary. Furthermore, since on the job the paste is often "burned," due to insufficient mixing, or is insufficiently slaked, it is apparent that slaking may be more efficiently done in an especially equipped plant than in the field. Consequently, despite the fact that quicklime works somewhat

more smoothly under the trowel, there has arisen within the past decade a considerable demand for hydrated lime.

**383. Hydrated Lime.**—When quicklime is finely crushed, completely slaked with a minimum amount of water and screened or ground to form a fine homogeneous powder the product is called hydrated lime or “limoid.” If the lime from which it is made is pure, hydrated lime is a white powder having a specific gravity of 2.08; the specific gravity of some dolomitic hydrates may reach 2.40. Hydrated lime is in general sold in paper bags of 40-lb. capacity or in 100-lb. burlap or cloth sacks. In such containers it may be stored for a much longer time than lump lime without serious deterioration.

**384. Testing of Limes.**—In addition to chemical analysis, the American Society for Testing Materials specifies a fineness test on slaked lime in order to determine the percentage of inert material in the product. For this test a representative 5-lb. sample of lime broken to pass a  $\frac{1}{4}$ -in. sieve, is carefully slaked with sufficient water to form the maximum amount of lime putty, care being taken to avoid drowning or burning. After standing for twenty-four hours the paste is washed through a 20-mesh sieve. The stream of water should flow under moderate pressure and rubbing of the material through the screen should be avoided. Not over 3 per cent of selected or 5 per cent of run-of-kiln lime should remain on the sieve.

Hydrated lime when tempered with water and formed into pats should pass the standard accelerated soundness test prescribed for Portland cement.

Tensile strength tests are also sometimes made on lime mortars. For this purpose the standard briquette used in cement testing is the form of specimen which is generally adopted. When compression or transverse tests are made it is necessary to specify a certain size of test-piece, since the rate of carbonation is affected by the thickness of the specimen. Large cubes have greater strengths than small ones. Standard methods of making strength tests have not been formulated.

The yield or volume of lime paste which can be made from a given weight of lime is an important factor in estimating the quantity of lime required to make a given amount of mortar. The test can be made in the same manner as indicated in Art. 463, Ch. XII.

**385. Properties of Lime.**—From a long series of experiments on the strength of lime-mortars made at Iowa State College the results shown in Table 1 have been derived. For some unknown reason the strength of these mortars was somewhat greater at nine months than at one year, although the carbonation process was still incomplete at the end of a year. In general the greater strengths were obtained with the lower percentages of water and sand. The hardening of the calcium limes was more uniform



and rapid than that of the magnesian limes, but the latter exhibited higher strengths.\*

TABLE 1.—RESULTS OF TENSILE TESTS OF LIME-MORTAR BRIQUETTES MADE AT IOWA STATE COLLEGE IN 1907

Proportions, Lime : Sand.*	% Water.†	AVERAGE STRENGTH OF 10 BRIQUETTES IN LB./IN.‡									
		Age 3 Months.					Age 6 Months.				
		Calcium Limes.		Magnesian Limes.			Calcium Limes.		Magnesian Limes.		
		Mason City.	Springfield.	Eagle Point.	Mason City.	Maquoketa.	Mason City.†	Springfield.	Eagle Point.	Mason City.	Maquoketa.
1 : 1	100	65	96	111	78	128	63	109	154	159	184
1 : 2		76	94	100	92	130	68	97	113	137	167
1 : 3		55	68	90	86	130	53	73	94	119	154
1 : 1	200	45	61	98	54	105	47	62	113	75	153
1 : 2		51	55	90	81	105	53	57	88	98	125
1 : 3		50	55	101	81	100	54	48	104	91	108
1 : 1	300	43	41	120	75	101	42	45	140	96	123
1 : 2		41	45	112	77	86	41	45	114	91	113
1 : 3		46	53	87	70	82	48	51	99	88	96

\* A river sand passing a No. 20 and held on a No. 30 sieve was used in all tests.

† Calculated in terms of the weight of the dry hydrate.

‡ Age = 16 weeks.

#### ANALYSES OF LIMES

Locality.	CaO	MgO	Al <sub>2</sub> O <sub>3</sub> +Fe <sub>2</sub> O <sub>3</sub>	Loss on Ignition.	Insol. Res.	CO <sub>2</sub>
Mason City, Ia. . . . .	95.40	0.43	2.98	0.00	1.02	trace
Springfield, Mo. . . . .	94.70	0.40	1.80	2.08	1.00	
Eagle Point, Ia. . . . .	58.19	33.48	6.60	slight	2.01	
Mason City, Ia. . . . .	72.40	15.23	6.03	3.36	2.32	0.10
Maquoketa, Ia. . . . .	60.60	35.70	2.10	2.30	0.63	

Compressive tests by W. E. Emley and S. E. Young, of the Bureau of Standards Laboratory, show that the size of the sand grains has a pronounced effect upon the strength of lime mortar.† The tests were made upon 2-in. cubes of 1 part quicklime to 3½ parts sand, by weight. The age at breaking was ninety days. The results of these tests appear in Table 2. It should be noted that the mortars of fine sand gave the high-

\* For further data see *Bulletin No. 1*, Vol. 4, Engineering Experiment Station, Iowa State College, Ames, Iowa; *Report of Chief Engineers*, U. S. A., 1896, Pt. 5, p. 2839; *Bulletin No. 4*, Ohio Geol. Survey; *Proc. A.S.T.M.*, Vol. 10, p. 328-40; also Vol. 14, p. 339.

† Reported in *Proc. A.S.T.M.*, Vol. 14, p. 346.

est strength, a condition which does not hold true in mortars made of hydraulic cements. Probably the loss in strength due to the lack of density in the fine-sand pastes is offset by the complete carbonation which is obtained on account of their porous structure.

The effect of the proportion of sand on the compressive strength of lime pastes is also shown in Table 2.

TABLE 2.—THE VARIATION IN THE COMPRESSIVE STRENGTH OF LIME WITH (a) AMOUNT OF SAND AND (b) SIZE OF SAND GRAINS

(Tests by Emley and Young)

Each result is averaged from 3 tests.

(a) AMOUNT OF SAND.			(b) SIZE OF GRAINS.		
Sand Parts.	Compressive Strength Lb. per Sq.in.		Size of Sand Grains between Meshes No.	Compressive Strength Lb. per Sq. in.	
	High Calcium	Dolomitic.		High Calcium.	Dolomite
1/2.....	273	372	10-20	98	166
1.....	151	267	20-30	118	214
2.....	116	217	30-40	138	312
3.....	112	202	40-60	186	335
4.....	116	203	60-80	260	444

High-calcium limes expand more in slaking and shrink more in setting than the magnesian limes. They are also more liable to injury through "burning" in slaking. The magnesian and dolomitic limes work more smoothly under the trowel and on account of slowness in hardening afford more time for surface treatment in wall plastering. Their slowness in hardening also permits a thorough hydration of the improperly burned particles before the mass has become rigid. Consequently, plaster made of magnesian lime is less likely to be disfigured by "lime pops" than that made of high calcium lime.

**386. The Uses of Lime.**—In construction, slaked lime is chiefly used to make mortar for laying brick and stone masonry and for plastering walls of buildings. When used for the latter purpose best results are obtained by slaking the lime for a period of three or four days. Hydrated lime is used for similar purposes but may be mixed into mortar and used immediately after arrival on the job. Being completely slaked, hydrated lime may be added to natural and Portland cement mortars to increase their plasticity and smoothness under the trowel. It is also used in making the popular stucco finish for exterior walls \* and to increase the imperviousness of Portland cement concrete (see Art. 533).

\* See specifications for Portland Cement Stucco, *Proc. N.A.C.U.* Vol. 7, p. 586.

**387. Hydraulic Lime.**—In the middle of the eighteenth century John Smeaton, the celebrated English engineer, was confronted with the problem of finding a cement which could be used in the construction of the famous Eddystone Lighthouse. The only cementing material then in use was quicklime, which does not harden under water. After a series of experiments he discovered that an impure limestone containing a small amount of clay, if calcined in the ordinary way, would produce a lime which would slake upon the addition of water and would harden under water. On account of the latter property the name hydraulic lime was given to this material. In France and southern Europe it is still used to a considerable extent. On account of the prevalence of raw materials suitable for the manufacture of Portland and natural cements no hydraulic lime is manufactured in the United States. However, Lafarge cement, a by-product in the manufacture of hydraulic lime, is used considerably in this country.

Hydraulic lime is manufactured in the same way as quicklime, although a somewhat higher temperature is required in burning. In slaking, considerable care is required to provide just sufficient water and no excess, since an excess would cause the lime to harden. After slaking, the coarse material is screened out and the fine product bagged for market. The coarse particles are finely ground and sold for natural cement. The specific gravity of hydraulic lime is about the same as that of the natural cement. Mortars made from the famous limes of Tiel, France, also have about the same strength as those made from natural cement.

**388. Lafarge Cement.**—This cement is used in America for stucco work and in laying marble and other masonry which is stained by natural or Portland cement mortars. It is a Grappier cement made as indicated above. Published analyses show it consists principally of lime (58–59 per cent) and silica (27–31 per cent) with 2.6 to 4.5 per cent of alumina and smaller percentages of iron oxide, magnesia and the alkalies. According to the manufacturers' circular \* this cement has a specific gravity of 2.6, an initial set in 4 hours, final set in 10 hours, and a residue on 100-mesh sieve of 0.6 per cent. Its strength neat and in a 1 : 2 mortar is about 60 per cent of that demanded of a standard Portland cement at 7 and 28 days. At two years the record gives a neat strength of 665 lb. per square inch.

## GYPSUM PLASTERS

**389. Introduction.**—On account of the wide use of gypsum plasters in the arts and in building construction a brief résumé of some important facts concerning them will be given. In the United States plaster of Paris, stucco, cement plaster, wall plaster, and hard finish plaster are

\* Eckel's *Cements, Limes and Plasters*, p. 185.

extensively used in wall construction. In Germany flooring-plaster, made by calcining gypsum at a high temperature, has been considerably used. In all of these powders, gypsum in a more or less dehydrated state is the essential element. When water is added to these substances they become rehydrated forming compounds similar to those existing before calcination.

**390. Gypsum.**—There are two commercial varieties of crude gypsum, rock gypsum and gypsum earth or gypsite. These substances consist principally of a hydrous sulphate of lime ( $\text{CaSO}_4 + 2\text{H}_2\text{O}$ ), with varying percentages of silica, carbonate of lime, carbonate of magnesia, and iron oxide. Pure gypsum is a white translucent crystalline mineral, so soft that it can be scratched with the finger-nail. When heated to  $400^\circ \text{F}$ ., pure gypsum loses its luster and its specific gravity is increased from 2.3 to approximately 2.95 due to the loss of water of crystallization.

Deposits of gypsum are numerous and widely scattered. The rock deposits occur in beds commonly traversed by thin strata of limestone and often adjacent to rock-salt deposits. Gypsite formations consist of masses of gypsum crystals interspersed with clay and sand. The states leading in the production of gypsum are New York, Iowa, Michigan, Ohio, Virginia, Texas and Oklahoma. Among the countries of the world the United States ranks first and produces about two-fifths of the total supply. The value of the 2,476,465 short tons produced in the United States in 1914 was \$6,895,989. Of the quantity obtained in this country about two-thirds is calcined and made into plasters; approximately one-sixth is sold in crude state to Portland cement manufacturers.

**391. Manufacture of Plasters.**—For making the refined grades of plaster of Paris in which a uniform degree of calcination is required the oven process is much used in Europe. In this country, however, on account of expense and time required, this method has been discarded in favor of the kettle and rotary process, the former being used in nearly all plants making plaster of Paris or cement plaster. Hard finish plasters are made in kilns similar to the mixed-feed kilns used in calcining lime.

After the raw material has been excavated or mined it is put through one or more crushers, and if the kettle process is used, is then ground by buhr-stones or like mills until about 60 per cent will pass a No. 100 sieve. In the rotary process the final pulverization is omitted until calcination is completed.

The kettles employed for calcination are 8 or 10 feet in diameter and about 6 or 7 feet high. The sides are made of sheet steel and the bottoms of cast iron. Each kettle is placed with its top just above the working floor. The lower portion of the kettle is enclosed in a masonry chamber which serves to support the kettle and distribute the heat from the grate fire about it. Two or four horizontal flues running through the kettle increase the circulation of heat. A power-driven stirrer is used to agitate

the contents of the kettle and thus prevent the bottoms from burning out. The hot gases and steam are led away through a stack.

After the pulverized material has been chuted into a kettle, heat is very slowly applied until the mechanically held water is driven off. At a temperature just above the boiling-point of water the whole mass bubbles up violently and then sinks. At 290° F. the combined water begins to boil out and between 340° and 396° F. the process is stopped. The impure gypsites generally require a higher temperature than the purer rock gypsums. In many plants the final temperature is judged by the appearance of the boil, although thermometers are often used in making plaster of Paris.

By the kettle process, it requires about two or three hours to calcine a charge yielding 5 or 6 tons. The calcined product is then run from the bottom of the kettle into a cooking vat, whence, after partially cooling, it is sent to the screens. Residues from the screens are reground; the fines are stored in large bins.

In the rotary process the raw material is crushed to pass a 1-in. mesh and is then fed into the upper end of a cylinder which rotates about an axis slightly inclined to the horizontal. Calcination is accomplished by the introduction of hot furnace gas, the temperature of which can be regulated by an admixture of a forced draught of air. When properly roasted the material is conveyed to brick-lined calcining vats in which further changes are brought about by the heat within the material. The product from the vats is then finely ground and screened. Continuous operation is the main advantage which this process possesses over the kettle process.

In order that the workman may properly handle plaster of Paris or stucco, it is necessary to delay the time of setting. This is accomplished by adding a fraction of one per cent of a retardant like glue, sawdust or blood after the plaster has cooled.

To increase the cohesiveness of wall plaster, cattle hair or wood fiber is introduced. For this purpose about 2 or 3 lb. of finely picked hair or 60 to 100 lb. of finely pulverized wood fiber is added to each ton of plaster.

Wall plasters made from pure raw materials are generally adulterated with 15 to 20 per cent of hydrated lime to increase the plasticity of the product. The term "stucco" is sometimes applied to a plaster so treated. Wall plasters made from raw materials containing considerable clay do not require such addition.

Many of the plasters are packed in jute sacks holding 100 lb. each. Recently a considerable effort has been made to further the use of paper sacks, thus obviating the traffic in empty jute bags, which is a considerable nuisance and expense.

**392. Plaster of Paris.**—Plaster of Paris is produced by incompletely

dehydrating pure, finely ground gypsum at a temperature somewhat less than 400° F. Most plasters closely approach the theoretical composition  $\text{CaSO}_4 + \frac{1}{2}\text{H}_2\text{O}$ —which contains about 6.2 per cent of water. Plaster of Paris is a white powder having a specific gravity of 2.57. When tempered with sufficient water to form a plastic paste it sets in 5 to 10 min.

The setting of plaster of Paris is attributed to the formation of gypsum crystals from a supersaturated aqueous solution. Why the rapidity of setting is so very much greater when the powder consists of plaster of Paris than when it consists of anhydrous gypsum plus water is a point not yet settled. When substances of a colloidal nature (glue for example) are mixed with the plaster the formation of crystals is hindered and the time of set retarded.

In hardening, plaster of Paris first shrinks and then expands. The latter property makes the material valuable in making casts, since a sharp impression of the mold can be secured. Owing to the rapidity of set and difficulty in working this material, its use in structures is limited to ornamental work.

**393. Cement Plaster.**—Cement plaster may be made by adding a retardant to plaster of Paris, or very commonly by using impure raw materials which produce a plaster that is slow setting. Ordinarily, less than three-fourths of one per cent of retardant is required. Some of the plasters made from impure materials are so slow in setting that small percentages of accelerators such as sodium chloride, sodium sulphate or sodium carbonate must be added to render the plaster usable.

Most plasters are ground so that 60 to 70 per cent will pass a 100-mesh sieve. Tests seem to indicate that the more finely ground plasters will produce mortars of highest strength. Results of a large number of tensile strength tests on plasters are given in *Rept. Iowa Geol. Surv.*, Vol. 12, p. 232. The plasters were hand mixed and packed into cement briquette molds by the fingers. About 35 per cent of water was used in making the neat specimens. Immediately after setting the specimens were removed from the molds and stored in air or water until tested. The results show that most of the neat plasters stored in air developed over one-half of their strength at one month in one day. Those specimens which were stored in water exhibited little increase in strength after one day and were weaker than those subjected to air-curing. The air-cured specimens gained but little strength after one month. The average strength of air-cured briquettes one month old made of nine different brands of plaster was 370 lb. per square inch.

Plaster and sand mortars of 1 : 1 proportions may be expected to develop tensile strengths of about 80 per cent of the neat strength at corresponding ages. Mortar of 1 : 2 proportions generally possess one-half to two-thirds of the neat strength.

Compressive strength tests on 2-in. plaster cubes by Slosson and Moody (*Tenth Annual Report Wyoming Agric. and Mech. Coll.*, 1900), furnish values of neat strength varying between 887 and 2236 lb./in.<sup>2</sup> The plasters were one week old and air cured. Those showing the highest strength had no retardant. Specimens of 1 : 1 plaster and sand mortars had about two-thirds of the strength of the neat test-pieces.

In the previously cited Iowa report Prof. Marston's tests showed that the adhesion of plaster mortars to a fractured surface of plaster was approximately two-thirds of the strength of the mortar.

Gypsum wall plasters have become quite popular in recent years because they are ready for use when brought to the job and also because they harden more rapidly than do the older lime plasters. However, lime plasters are more plastic and may, therefore, be loaded with 3 or 4 parts of sand, whereas the gypsum plasters cannot carry over 2 parts; also, when properly slaked, lime plasters form eventually just as satisfactory walls as those made from cement plasters.

**394. Hard Finish Plasters.**—By burning gypsum to a considerably higher temperature than the calcining temperature of cement plaster, treating with certain solutions like alum and Glauber's salts and burning again, there may be produced plasters which set slowly but ultimately become very hard. Such plasters may be polished to form a smooth surface and make a very satisfactory finish for interior walls. Often walls of these plasters are marked to imitate tiling with pleasing results.

*Keene's cement* is made by burning a very pure rock gypsum at a cherry-red heat, then dipping the calcined product into a 10 per cent solution of alum and burning again at a red heat. Subsequently the material is finely ground. Keene's cement is practically pure calcium sulphate ( $\text{CaSO}_4$ ) with a very small percentage of calcium carbonate ( $\text{CaCO}_3$ ). On account of the care taken in selecting the raw material the plaster is of an unusually pure white color. At seven days its neat strength may be expected to be between 500 and 700 lb.

Mack's cement is made by burning gypsum at a very high temperature and adding about 0.4 per cent of burnt Glauber's salts ( $\text{Na}_2\text{SO}_4$ ) or potassium sulphate ( $\text{K}_2\text{SO}_4$ ). It is said to form an unusually hard dense and durable surface which will take paint well.

**395. Other Gypsum Building Materials.**—Blocks and tile made from wall plaster and suitable for floor and interior wall construction are now on the market. These forms are light, can be easily sawn to desired shape, possess sufficient strength for many types of construction and have a high resistance to fire.\* Mixed with fine cinders or wood chips and sufficient water to form a thin consistency, wall plaster has been

\* See *Gypsum as a Fireproofing Material* by V. G. Marani, *Jour. Cleveland Engr. Soc.*, Vol. 7, p. 213.

used in making floors for buildings. Such floors can be much more rapidly constructed than concrete floors, owing to the rapidity with which the plaster hardens.\* However, they are neither so resistant to fire nor so strong as concrete floors. Another product of recent origin is plaster board. It is made of thin layers of cardboard or wood cemented together by wall plaster. It is used in place of lath or it may be used in place of plaster walls by covering the joints with neatly finished wooden strips.

\* See *Bulletin* No. 25, Public Works of the Navy, Jan. 1917.



## CHAPTER XII

### METHODS OF TESTING HYDRAULIC CEMENTS

**396. Necessity for Testing Cement.**—Experience has shown that it is practically impossible to make large quantities of cement without any variation in quality. To be sure, some mills working with raw materials which run very uniformly and using the best of equipment and methods of operation will have very few unsuccessful “burns” in a year, while others will be less fortunate. Nevertheless the consumer has little chance of ascertaining how his particular carload of cement was made; therefore, if he has under way a construction of any importance, he ought to satisfy himself regarding the quality of his purchase. He should test his cement not only to see that he gets what he has paid for but also to forestall the possibility of a failure through the use of defective material.

In engineering construction the main qualifications demanded of a cement are permanency of structure, strength, and a rate of setting suitable to the demands of the work. To determine these qualifications, both physical and chemical tests are made, the former, on account of importance, more often than the latter.

As a result of long experience the physical tests which have come into general use in determining the acceptability of cement are: (1) soundness or constancy of volume, (2) strength, (3) time of set or activity, (4) fineness, and (5) specific gravity. In order that the results of such tests made by different parties may accord as nearly as possible, it is necessary that a standard method be rigidly adhered to and that only experienced operators, who fully appreciate the necessity of eliminating personal equation from all manipulations, be employed.

In the fine type of the following sections will be considered the Standard Specifications and Tests for Portland Cement as revised by the American Society for Testing Materials (effective Jan. 1, 1917). The methods of testing are adapted to any of the hydraulic cements with the exception of the accelerated soundness test which is used with Portland cements only. The sections of the standard methods will be numbered consecutively but will be interspersed with comments and references to other methods of testing which will appear in large type. Since it is only by close observance of standard methods that uniformity in testing may obtain, any divergence by the operator from such methods should be stated with full explanation in reporting the results of tests.

## STANDARD SPECIFICATIONS AND TESTS FOR PORTLAND CEMENT \*

1917

These specifications are the result of several years' work of a special committee representing a United States Government Departmental Committee, the Board of Direction of the American Society of Civil Engineers, and Committee C-1 on Cement of the American Society for Testing Materials in co-operation with Committee C-1.

## SPECIFICATIONS

1. **Definition.**—Portland cement is the product obtained by finely pulverizing clinker produced by calcining to incipient fusion an intimate and properly proportioned mixture of argillaceous and calcareous materials, with no additions subsequent to calcination excepting water and calcined or uncalcined gypsum.

## I. CHEMICAL PROPERTIES

2. **Chemical Limits.**—The following limits shall not be exceeded:

Loss on ignition, per cent. ....	4.00
Insoluble residue, per cent. ....	0.85
Sulphuric anhydride (SO <sub>3</sub> ), per cent. ....	2.00
Magnesia (MgO), per cent. ....	5.00

## II. PHYSICAL PROPERTIES AND TESTS

3. **Specific Gravity.**—The specific gravity of cement shall be not less than 3.10 (3.07 for white Portland cement). Should the test of cement as received fall below this requirement a second test may be made upon an ignited sample. The specific gravity test will not be made unless specifically ordered.

4. **Fineness.**—The residue on a standard No. 200 sieve shall not exceed 22 per cent by weight.

5. **Soundness.**—A pat of neat cement shall remain firm and hard, and show no signs of distortion, cracking, checking, or disintegration in the steam test for soundness.

6. **Time of Setting.**—The cement shall not develop initial set in less than 45 minutes when the Vicat needle is used or 60 minutes when the Gillmore needle is used. Final set shall be attained within 10 hours.

7. **Tensile Strength.**—The average tensile strength in pounds per square inch of not less than three standard mortar briquettes (see Section 51) composed of one part of cement and three parts standard sand, by weight, shall be equal to or higher than the following:

Age at Test, Days.	Storage of Briquettes.	Tensile Strength, Lb. per Sq. In.
7	1 day in moist air, 6 days in water. ....	200
28	1 day in moist air, 27 days in water. ....	300

8. The average tensile strength of standard mortar at 28 days shall be higher than the strength at seven days.

\* See *Year Book of A.S.T.M.* for 1916, serial designation C9-17. The United States Government has also adopted substantially the same specifications.

## III. PACKAGES, MARKING AND STORAGE

9. **Packages and Marking.**—The cement shall be delivered in suitable bags or barrels with the brand and name of the manufacturer plainly marked thereon, unless shipped in bulk. A bag shall contain 94 lb. net. A barrel shall contain 376 lb. net.

10. **Storage.**—The cement shall be stored in such a manner as to permit easy access for proper inspection and identification of each shipment, and in a suitable weather-tight building which will protect the cement from dampness.

## IV. INSPECTION

11. **Inspection.** Every facility shall be provided the purchaser for careful sampling and inspection at either the mill or at the site of the work, as may be specified by the purchaser. At least 10 days from the time of sampling shall be allowed for the completion of the 7-day test, and at least 31 days shall be allowed for the completion of the 28-day test. The cement shall be tested in accordance with the methods hereinafter prescribed. The 28-day test shall be waived only when specifically so ordered.

## V. REJECTION

12. **Rejection.**—The cement may be rejected if it fails to meet any of the requirements of these specifications.

13. Cement shall not be rejected on account of failure to meet the fineness requirement if upon retest after drying at 100° C. for one hour it meets this requirement.

14. Cement failing to meet the tests for soundness in steam may be accepted if it passes a retest using a new sample at any time within 28 days thereafter.

15. Packages varying more than 5 per cent from the specified weight may be rejected; and if the average weight of packages in any shipment, as shown by weighing 50 packages taken at random, is less than that specified, the entire shipment may be rejected.

## TESTS

## SAMPLING

## 397. A. S. T. M. Method.—

16. **Number of Samples.**—Tests may be made on individual or composite samples as may be ordered. Each test sample should weigh at least 8 lb.

17. (a) *Individual Sample.*—If sampled in cars one test sample shall be taken from each 50 bbl. or fraction thereof. If sampled in bins one sample shall be taken from each 100 bbl.

(b) *Composite Sample.*—If sampled in cars one sample shall be taken from one sack in each 40 sacks (or 1 bbl. in each 10 bbl.) and combined to form one test sample. If sampled in bins or warehouses one test sample shall represent not more than 200 bbl.

18. **Method of Sampling.**—Cement may be sampled at the mill by any of the following methods that may be practicable, as ordered:

(a) *From the Conveyor Delivering to the Bin.*—At least 8 lb. of cement shall be taken from approximately each 100 bbl. passing over the conveyor.

(b) *From Filled Bins by Means of Proper Sampling Tubes.*—Tubes inserted vertically may be used for sampling cement to a maximum depth of 10 ft. Tubes inserted horizontally may be used where the construction of the bin permits. Samples shall be taken from points well distributed over the face of the bin.

(c) *From Filled Bins at Points of Discharge.*—Sufficient cement shall be drawn from the discharge openings to obtain samples representative of the cement contained in the bin, as determined by the appearance at the discharge openings of indicators placed on the surface of the cement directly above these openings before drawing of the cement is started.

19. *Treatment of Sample.*—Samples preferably shall be shipped and stored in air-tight containers. Samples shall be passed through a sieve having 20 meshes per linear inch in order to thoroughly mix the sample, break up lumps and remove foreign materials.

398. *Selection of the Sample.*—Bearing in mind the importance of obtaining a fair and representative sample of the cement, it is well to exercise much care in the selection, storage, and mixing of it. If the cement is stored in bags in a car or warehouse the selection of the bags to be sampled should be made in such manner that the entire lot will be represented. It is not sufficient to select bags from the outside of a pile, but those within should also be sampled.

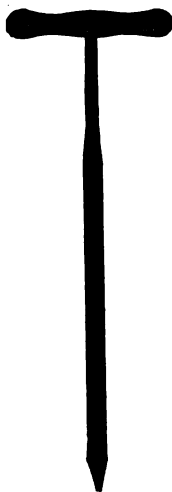


FIG. 1.—Cement Sampler.

In all cases a sufficient quantity should be collected so that the contemplated series of tests may be duplicated without resampling. Although it is not always possible to obtain a second sample; yet it is worth while in sampling bags or barrels, to number the bag and sample alike. The sample may be taken conveniently by the auger shown in Fig. 1 or by a long-handled spoon.

399. *Storage of the Sample.*—Because it is important that the properties of the cement shall not be influenced by changes in the humidity and temperature of the air, standard storage conditions should be maintained. If paper or cloth sacks are used in collecting samples, they should be enclosed in a tight waterproof container for shipment to the laboratory. In the laboratory, clean cans fitted with tightly fitting covers will be found convenient receptacles for samples which must be stored for several weeks. The storage room should be dry and its temperature maintained at approximately 70° F.

400. *Mixing Samples.*—If information concerning the uniformity of a quantity of cement is desired, the individual samples should be tested separately. If a fair number of samples are taken from a car load, say ten to twenty, such procedure would be very costly if all the tests were made on each sample. In order to cut down the expense of testing and at the same time secure information regarding the uniformity of the material, the accelerated soundness may be run on every sample and the remaining tests made on a composite sample consisting of equal portions taken from each individual sample. Before making any tests, each individual or composite sample should be placed on a smooth dense surface and turned over 30 or 40 times with a trowel.

**401. Quartering.**—The portions of a sample required for the various tests may be fairly obtained by spreading the cement into a flat cylindrical pile and removing sector slices sufficient in quantity for each test. Such proceeding is sometimes called "the method of quartering."

#### CHEMICAL ANALYSIS

**402. Purpose.**—Chemical analyses are regularly made by the manufacturer to whom they afford very valuable indications concerning the proportioning and burning of the cement. Sometimes analyses are made by consumers who wish to determine whether the cement has been adulterated or has excessive amounts of injurious substances such as magnesia and sulphur trioxide. As a rule the results of an analysis are of much less value to the consumer than the physical test indications. Only the methods for the more important determinations appear in the following articles; for more complete instructions see *Meade's Portland Cement*, Chapter X.

#### **403. A. S. T. M. Method for Finding Loss on Ignition.**

**20. Method.**—One gram of cement shall be heated in a weighed covered platinum crucible, of 20 to 25-cc. capacity, as follows, using either method (a) or (b) as ordered:

(a) The crucible shall be placed in a hole in an asbestos board, clamped horizontally so that about three-fifths of the crucible projects below, and blasted at a full red heat for 15 minutes with an inclined flame; the loss in weight shall be checked by a second blasting for 5 minutes. Care shall be taken to wipe off particles of asbestos that may adhere to the crucible when withdrawn from the hole in the board. Greater neatness and shortening of the time of heating are secured by making a hole to fit the crucible in a circular disc of sheet platinum and placing this disc over a somewhat larger hole in an asbestos board.

(b) The crucible shall be placed in a muffle at any temperature between 900 and 1000° C. for 15 minutes and the loss in weight shall be checked by a second heating for 5 minutes.

**21. Permissible Variation.**—A permissible variation of 0.25 will be allowed, and all results in excess of the specified limit but within this permissible variation shall be reported as 4 per cent.

#### **404. A. S. T. M. Method for Determining Insoluble Residue.**

**22. Method.**—To a 1-g. sample of cement shall be added 10 cc. of water and 5 cc. of concentrated hydrochloric acid; the liquid shall be warmed until effervescence ceases. The solution shall be diluted to 50 cc. and digested on a steam bath or hot plate until it is evident that decomposition of the cement is complete. The residue shall be filtered, washed with cold water, and the filter paper and contents digested in about 30 cc. of a 5 per cent solution of sodium carbonate, the liquid being held at a temperature just short of boiling for 15 minutes. The remaining residue shall be filtered, washed with cold water, then with a few drops of hot hydrochloric acid, 1 : 9, and finally with hot water, and then ignited at a red heat and weighed as the insoluble residue.

**23. Permissible Variation.**—A permissible variation of 0.15 will be allowed, and

all results in excess of the specified limit but within this permissible variation shall be reported as 0.85 per cent.

#### 405. A. S. T. M. Method for Determining Sulphuric Anhydride.

**24. Method.**—One gram of the cement shall be dissolved in 5 cc. of concentrated hydrochloric acid diluted with 5 cc. of water, with gentle warming; when solution is complete 40 cc. of water shall be added, the solution filtered, and the residue washed thoroughly with water. The solution shall be diluted to 250 cc., heated to boiling and 10 cc. of a hot 10 per cent solution of barium chloride shall be added slowly, drop by drop, from a pipette and the boiling continued until the precipitate is well formed. The solution shall be digested on the steam bath until the precipitate has settled. The precipitate shall be filtered, washed, and the paper and contents placed in a weighed platinum crucible and the paper slowly charred and consumed without flaming. The barium sulphate shall then be ignited and weighed. The weight obtained multiplied by 34.3 gives the percentage of sulphuric anhydride. The acid filtrate obtained in the determination of the insoluble residue may be used for the estimation of sulphuric anhydride instead of using a separate sample.

**25. Permissible Variation.**—A permissible variation of 0.10 will be allowed, and all results in excess of the specified limit but within this permissible variation shall be reported as 2.00 per cent.

#### 406. A. S. T. M. Method for Determining Magnesia.

**26. Method.**—To 0.5 g. of the cement in an evaporating dish shall be added 10 cc. of water to prevent lumping and then 10 cc. of concentrated hydrochloric acid. The liquid shall be gently heated and agitated until attack is complete. The solution shall then be evaporated to complete dryness on a steam or water bath. To hasten dehydration the residue may be heated to 150° or even 200° C. for one-half to one hour. The residue shall be treated with 10 cc. of concentrated hydrochloric acid diluted with an equal amount of water. The dish shall be covered and the solution digested for ten minutes on a steam bath or water bath. The diluted solution shall be filtered and the separated silica washed thoroughly with water.\* Five cubic centimeters of concentrated hydrochloric acid and sufficient bromine water to precipitate any manganese which may be present, shall be added to the filtrate (about 250 cc.). This shall be made alkaline with ammonium hydroxide, boiled until there is but a faint odor of ammonia, and the precipitate, iron and aluminum hydroxides, after settling, shall be washed with hot water, once by decantation and slightly on the filter. Setting aside the filtrate, the precipitate shall be transferred by a jet of hot water to the precipitating vessel and dissolved in 10 cc. of hot hydrochloric acid. The paper shall be extracted with acid, the solution and washings being added to the main solution. The aluminum and iron shall then be reprecipitated at boiling heat by ammonium hydroxide and bromine water in a volume of about 100 cc., and the second precipitate shall be collected and washed on the filter used in the first instance if this is still intact. To the combined filtrates from the hydroxides of iron and aluminum, reduced in volume if need be, 1 cc. of ammonium hydroxide shall be added, the solution brought to boiling, 25 cc. of a saturated solution of boiling ammonium oxalate added, and the boiling continued until the precipitated calcium oxalate has assumed a well-defined granular form. The precipitate after one hour shall be filtered and washed, then with the filter shall be placed wet in a platinum crucible, and the paper burned off over a small flame of a Bunsen burner; after ignition it shall be redissolved in hydrochloric acid and the solution diluted to 100 cc. Ammonia shall be added in slight excess,

\* Since this procedure does not involve the determination of silica, a second evaporation is unnecessary.

and the liquid boiled. The lime shall then be reprecipitated by ammonium oxalate, allowed to stand until settled, filtered and washed. The combined filtrates from the calcium precipitates shall be acidified with hydrochloric acid, concentrated on the steam bath to about 150 cc., and made slightly alkaline with ammonium hydroxide, boiled and filtered (to remove a little aluminum and iron and perhaps calcium). When cool, 10 cc. of saturated solution of sodium-ammonium-hydrogen phosphate shall be added with constant stirring. When the crystallin ammonium-magnesium orthophosphate has formed, ammonia shall be added in moderate excess. The solution shall be set aside for several hours in a cool place, filtered and washed with water containing 2.5 per cent of  $\text{NH}_3$ . The precipitate shall be dissolved in a small quantity of hot hydrochloric acid, the solution diluted to about 100 cc., 1 cc. of a saturated solution of sodium-ammonium-hydrogen phosphate added, and ammonia drop by drop, with constant stirring, until the precipitate is again formed as described and the ammonia is in moderate excess. The precipitate shall then be allowed to stand about two hours, filtered and washed as before. The paper and contents shall be placed in a weighed platinum crucible, the paper slowly charred, and the resulting carbon carefully burned off. The precipitate shall then be ignited to constant weight over a Meker burner, or a blast not strong enough to soften or melt the pyrophosphate. The weight of magnesium pyrophosphate obtained multiplied by 72.5 gives the percentage of magnesia. The precipitate so obtained always contained some calcium and usually small quantities of iron, aluminum, and manganese as phosphates.

**27. Permissible Variation.**—A permissible variation of 0.4 will be allowed, and all results in excess of the specified limit but within this permissible variation shall be reported as 5.00 per cent.

**407. Purity Test.**—The so-called purity test affords a speedy determination of the presence of adulterants. The test may be made as follows: Place about one-half teaspoonful of cement in a porcelain or glass vessel preferably in a test-tube. Stir the cement with a glass rod and cover with a 50 per cent solution of hydrochloric acid. If the cement is pure it will effervesce slightly and form a yellow or orange-yellow jelly; the presence of an adulterant consisting mainly of calcium carbonate will cause violent effervescing. Slag will cause the emission of sulphuretted hydrogen; other insoluble adulterants form a dark sediment at the bottom of the jelly.

#### DETERMINATION OF SPECIFIC GRAVITY

##### 408. A. S. T. M. Method.

**28. Apparatus.**—The determination of specific gravity shall be made with a standardized Le Chatelier apparatus which conforms to the requirements illustrated in Fig 2. This apparatus is standardized by the United States Bureau of Standards. Kerosene free from water, or benzine not lighter than 62° Baumé, shall be used in making this determination.

**29. Method.**—The flask shall be filled with either of these liquids to a point on the stem between zero and one cubic centimeter, and 64 g. of cement, of the same temperature as the liquid, shall be slowly introduced, taking care that the cement does not adhere to the inside of the flask above the liquid and to free the cement from air by rolling the flask in an inclined position. After all the cement is introduced, the level of the liquid will rise to some division of the graduated neck; the difference between readings is the volume displaced by 64 g. of the cement.

The specific gravity shall then be obtained from the formula

$$\text{Specific gravity} = \frac{\text{Weight of cement (g.)}}{\text{Displaced volume (cc.)}}$$

30. The flask, during the operation, shall be kept immersed in water, in order to avoid variations in the temperature of the liquid in the flask, which shall not exceed  $0.5^{\circ}\text{C}$ . The results of repeated tests should agree with 0.01.

31. The determination of specific gravity shall be made on the cement as received; if it should fall below 3.10, a second determination shall be made after igniting the sample as described in Section 20.

### FINESS

#### 409. A. S. T. M. Method.

32. Apparatus.—Wire cloth for standard sieves for cement shall be woven (not twilled) from brass, bronze, or other suitable wire, and mounted without distortion on frames not less than  $1\frac{1}{2}$  in. below the top of the frame. The sieve frames shall be circular, approximately 8 in. in diameter, and may be provided with a pan and cover.

33. A standard No. 200 sieve is one having nominally an 0.0029-in. opening and 200 wires per inch standardized by the U. S. Bureau of Standards, and conforming to the following requirements:

The No. 200 sieve should have 200 wires per inch, and the number of wires in any whole inch shall not be outside the limits of 192 to 208. No opening between adjacent parallel wires shall be more than 0.0050 in. in width. The diameter of the wire should

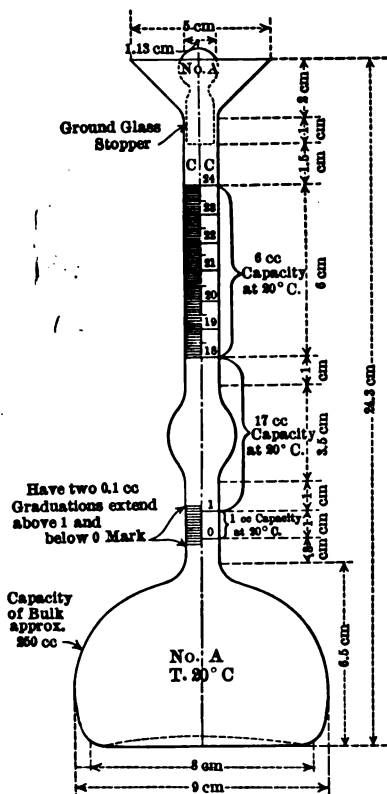


Fig. 2.—Le Chatelier Apparatus.\*

be 0.0021 in. and the average diameter shall not be outside the limits 0.0019 to 0.0023 in. The value of the sieve as determined by sieving tests made in conformity with the standard specification for these tests on a standardized cement which gives a residue of 25 to 20 per cent on the No. 200 sieve, or on other similarly graded material, shall not show a variation of more than 1.5 per cent above or below the standards maintained at the Bureau of Standards.

34. Method.—The test shall be made with 50 g. of cement. The sieve shall be thoroughly clean and dry. The cement shall be placed on the No. 200 sieve, with pan and cover attached, if desired, and shall be held in one hand in a slightly inclined position so that the sample will be well distributed over the sieve, at the same time gently striking the side about 150 times per minute against the palm of the other hand on the up stroke. The sieve shall be turned every 25 strokes about one-sixth of a

\* Several other forms of satisfactory apparatus for determining the specific gravity of cement are described in Meade's *Portland Cement*, Ch. XIV.



revolution in the same direction. The operation shall continue until not more than 0.05 g. passes through in one minute of continuous sieving. The fineness shall be determined from the weight of the residue on the sieve expressed as a percentage of the weight of the original sample.

35. Mechanical sieving devices may be used, but the cement shall not be rejected if it meets the fineness requirement when tested by the hand method described in Section 34.

36. **Permissible Variation.**—A permissible variation of 1 will be allowed and all results in excess of the specified limit but within this permissible variation shall be reported as 22 per cent.

**410. Precautions in Sieving.**—In making the fineness test, great care should be exercised by the operator to prevent loss of fine powder. Trial weighings may be made upon the material caught in the pan but the final weighing should always be made upon the material held on the sieve. If trial weighings are to be accurately made, the scales employed should be accurate and sensitive to 0.01 gram.

**411. Mechanical Shakers.**—In spite of the fact that hand sieving is given preference in the specifications a number of mechanical sifting devices have been described and illustrated in technical periodicals in recent years, some of which may well be employed by laboratories where a large amount of experimental work is being performed. (See *Engr. News*, Vol. 62, p. 728; *Proc. A.S.T.M.*, Vol. 10, p. 572.)

In operating such shakers it is customary to sift the cement for a certain time at a given rate. This method does not give results which agree with those recommended by standard specifications since different cements will pass through the sieves at different time rates.

If the method recommended by the A. S. T. M. for hand shaking be followed, little time will be gained unless the shaker is made to accommodate several sets of sieves and pans. A shaker which can be readily modified to include a larger number of sieves



FIG. 3.—A Ro-tap Mechanical Shaker Equipped with a Stop-Rite Time Switch.

is described by Mr. A. D. Gates in the *Engr. News*, Vol. 66, p. 446.

A shaker recently developed by the W. S. Tyler Co. of Cleveland is shown in Fig. 3. This device is so designed that six 8-in. standard sieves or 13 half-height sieves may be shaken at one operation. The driving mechanism is arranged to rotate the sieves and tap them 150 times a minute. By using the time switch, shown at the right, the shaker may be automatically stopped at the end of a predetermined period. Great accuracy and reliability are claimed for this shaker.

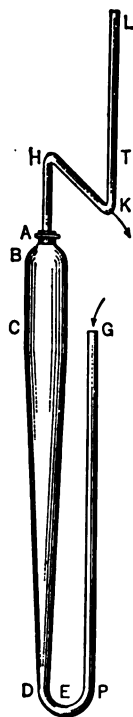


FIG. 4. — The Schöne Washing Apparatus.

**412. Other Methods of Determining Fineness.**—Since it is impossible to make sieves of sufficiently small mesh to determine the sizes of the very fine flour in the cement (the part which exerts the greatest influence on cementitious properties of the cement), recourse is often made to liquid or air methods of separation. A brief description of the Schöne Washing apparatus follows.

The essential part of the Schöne apparatus is shown in Fig. 4. It consists of a bent glass tube, conical from *D* to *C*, but cylindrical from *C* to *B*. The tube *AHKL* is also a glass tube, having an escape opening at *K*. The material to be assorted is placed in the conical portion *CD*, and the washing liquid is introduced at *G* and escapes at *K*, through an orifice about  $\frac{1}{16}$  in. in diameter, under any fixed head *KT*, controlled by the rate of admission at *G*. This head *KT* is the argument which is read on the graduated tube when in use to give the rate of flow, and hence the rate of upward velocity in the cylindrical portion *CB*. When properly standardized, the upward movement in this portion can be read from a diagram, or table, in terms of the head *KT*, which can be made a meter or more. The apparatus is defective as a quantitative separator, since there is no means of stirring the material in the conical part of the tube, and hence some of those particles which could readily be carried over for any particular upward velocity in *CB* will remain entangled in the mass of material left in the conical part *DC*.

Dr. W. Michaelis found the relation between the largest diameter of particle and the rate of upward flow for absolute alcohol and Portland cement to be  $d = 0.036v^{\frac{1}{2}}$ , where  $d$  = largest diameter in millimeters, and  $v$  = upward velocity of flow in millimeters per second in the cylindrical part of the washing apparatus.

By using coal-oil on Portland cement the *average* diameters of the particles carried over for different velocities in *CB* very nearly agree with the above formula for absolute alcohol and the largest diameters. Since

these particles are quite angular, the word diameter is here used as the mean transverse dimension as measured on a microscope-scale.

In some work carried out in the preparation of a thesis \* under the direction of J. B. Johnson two samples of Portland cement, one American and the other German, both of standard manufacture, were graded into five sizes by this method after having passed a No. 150 sieve. The sizes and the corresponding velocities of upward flow for each grade are given in the following table, and the percentages the weights of each grade were of the entire sample. This table shows that only about 40 per cent of the cement was of the finest grade, and probably not more than half of this was of the impalpable powder which really composes the active portion of the cement.

None of the other grades showed any tendency to harden after washing thoroughly with gasoline, drying, and wetting with water, whereas the finest grade showed its normal hardening properties when so treated.

#### GRADUATION OF PORTLAND CEMENT BY THE SCHÖNE APPARATUS

Brand of Cement.	PERCENTAGES OF ORIGINAL SAMPLES.						
	Carried by a Velocity of 0.38 mm. per Second. Maximum Diameter less than 0.02 mm.	Carried by a Velocity of 1.00 mm. per Second. Mean Diameter of 0.030 mm.	Carried by a Velocity of 1.78 mm. per Second. Mean Diameter of 0.048 mm.	Carried by a Velocity of 2.78 mm. per Second. Mean Diameter of 0.068 mm.	Carried by a Velocity of 4.00 mm. per Second. Mean Diameter of 0.097 mm.	Passed No. 150 Sieve and resisted a Velocity of 4 mm. per Second. Mean Diameter of 0.116 mm.	Held on a No. 150 Sieve. Mean Diameter of 0.180 mm.
Atlas (Amer.).	45	6	9	8	7	9	16
Star (Stettin, Germany) . .	36	7	9	8	8	9	23

One of the best known air separators is the Gary-Lindner apparatus which is described in the *Proc. I. A. T. M.*, 5th Congress 1908, 107. The United States Bureau of Standards Laboratory has also perfected an air sifting device (see *Technologic Paper* No. 48).

Feret has recently described a method for distinguishing the relative fineness of different cements by determining the rate at which they absorb moisture from humid air (see *Revue Des Mat. de Const. et de Trav. Publics*, 1913; or *Concrete and Constr. Engr.*, Vol. 8, p. 721).

For other methods of separating very fine powders see Wiley's *Principles and Practice of Agricultural Analysis*, Vol. 1, p. 200, etc.; Meade's *Portland Cement*, p. 390; *Proc. I. A. T. M.*, 5th Congress, 1909; 10<sub>8</sub>; also 6th Congress, 1912, 15<sub>1</sub>, 15<sub>2</sub>, 15<sub>3</sub>, 15<sub>4</sub>.

\* By Edward Conzelman and F. A. Rapp.

## MIXING CEMENT PASTES AND MORTARS

## 413. A. S. T. M. Method.

37. Method.—The quantity of dry material to be mixed at one time shall not exceed 1000 g. nor be less than 500 g. The proportions of cement or cement and sand shall be stated by weight in grams of the dry materials; the quantity of water shall be expressed in cubic centimeters (1 g. = 1 cc.). The dry materials shall be weighed, placed upon a non-absorbent surface, thoroughly mixed dry if sand is used, and a crater formed in the center, into which the proper percentage of clean water shall be poured; the material on the outer edge shall be turned into the crater by the aid of a trowel. After an interval of  $\frac{1}{2}$  minute for the absorption of the water, the operation shall be completed by continuous, vigorous mixing, squeezing and kneading with the hands for at least one minute.\* During the operation of mixing the hands, should be protected by rubber gloves.

38. The temperature of the room and the mixing water shall be maintained as nearly as practicable at 21° C. (70° F.).

414. Additional Recommendations.—Only water which is known to be pure should be used in mixing pastes and mortars. For the effect of variations in the temperature of mixing water see the report of Committee on Testing Cement and Cement Products, *Proc. Nat. Asso. Cement Users*, Vol. 5, p. 484. In weighing the requisite amounts of materials, scales accurate and sensitive to one-half gram should be used. Fig. 5 shows a scales suitable for this purpose.

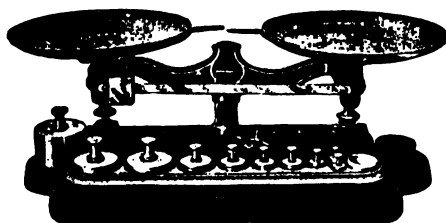


FIG. 5.—Balance for Weighing Cement and Sand.

415. Kneading.—The practice frequently employed in kneading is as follows: The operator covers the pile of material with his hands

placed palms downward with fingers pointing away from him. He then quickly forces the wrists forward keeping the ends of the fingers fixed and exerting a downward pressure of about 30 lb. with the palms. After three or four similar movements the pile is turned through an angle of 90 degrees, any loose material gathered in and the above manipulation repeated. The pile should be given twenty to twenty-five turns in a minute.

## NORMAL CONSISTENCY

## 416. A. S. T. M. Method.—

39. Apparatus.—The Vicat apparatus consists of a frame *A* (Fig. 6), bearing a movable rod *B*, weighing 300 g., one end *C* being 1 cm. in diameter for a distance of 6 cm., the other having a removable needle *D*, 1 mm. in diameter, 6 cm. long. The

\* In order to secure uniformity in the results of tests for the time of setting and tensile strength the manner of mixing above described should be carefully followed. At least one minute is necessary to obtain the desired plasticity which is not appreciably affected by continuing the mixing for several minutes. The exact time necessary is dependent upon the personal equation of the operator. The error in mixing should be on the side of over mixing.

rod is reversible, and can be held in any desired position by a screw *E*, and has midway between the ends a mark *F* which moves under a scale (graduated to millimeters) attached to the frame *A*. The paste is held in a conical, hard-rubber ring *G*, 7 cm. in diameter at the base, 4 cm. high, resting on a glass plate *H* about 10 cm. square.

**40. Method.**—In making the determination, 500 g. of cement with a measured quantity of water, shall be kneaded into a paste, as described in Section 37, and quickly formed into a ball with the hands, completing the operation by tossing it six times from one hand to the other, maintained about 6 in. apart; the ball resting in the palm of one hand shall be pressed into the larger end of the rubber ring held in the other hand, completely filling the ring with paste; the excess at the larger end shall then be removed by a single movement of the palm of the hand; the ring shall then be placed on its larger end on a glass plate and the excess paste at the smaller end, sliced off at the top of the ring by a single oblique stroke of a trowel held at a slight angle with the top of the ring. During these operations care shall be taken not to compress the paste. The paste confined in the ring resting on the plate, shall be placed under the rod the larger end of which shall be brought in contact with the surface of the paste; the scale shall be then read, and the rod quickly released. The paste shall be of normal consistency when the cylinder settles to a point 10 mm. below the original surface in 1 minute after being released. The apparatus shall be free from all vibrations during the test. This paste shall be made with varying percentages of water until the normal consistency is obtained. The amount of water required shall be expressed in percentage of weight of the dry cement.

5. The consistency of standard mortar shall depend upon the amount of water required to produce a paste of normal consistency from the cement. Having determined the normal consistency of the sample, all mortar made from the same sample shall be made with the same amount of water being expressed in percentage of the combined dry weights of the cement and sand.

**41. The Ball Method.**—Since by the Vicat method the normal consistency of a sample of cement is determined, the normal consistency of a sample of cement may be determined by the Bougué method. The test is made as follows: Employing a trowel, molding a ball of cement in the form of a 2-in. ball, the ball shall be dropped from a height of 2 ft. upon the

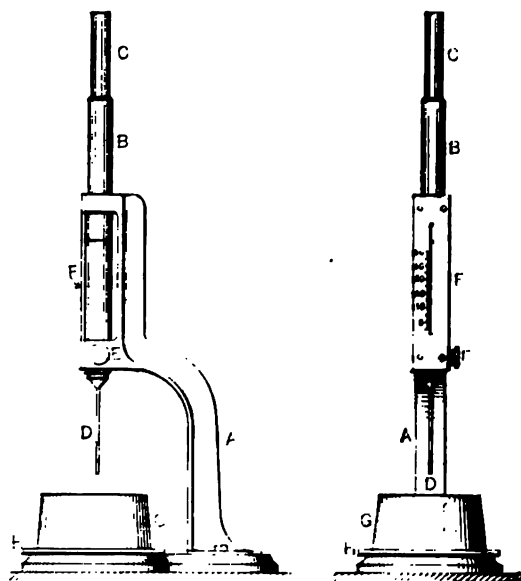


FIG. 6.—Vicat Apparatus

TABLE 1.—PERCENTAGE OF WATER FOR STANDARD MORTARS

Percentage of Water for Neat Cement Paste of Normal Consistency.	Percentage of Water for One Cement, Three Standard Ottawa Sand.	Percentage of Water for Neat Cement Paste of Normal Consistency.	Percentage of Water for One Cement, Three Standard Ottawa Sand.
15	9.0	23	10.3
16	9.2	24	10.5
17	9.3	25	10.7
18	9.5	26	10.8
19	9.7	27	11.0
20	9.8	28	11.2
21	10.0	29	11.3
22	10.2	30	11.5

obtains when the ball does not crack and does not flatten more than one-half its original diameter.

**418. Feret's Consistency Formula.**—The percentages of water given for standard mortars was derived from Feret's formula  $w = \frac{2}{3} \frac{p}{s+1} + k$ , in which  $w$  is the percentage of water required for mortar,  $p$  is the percentage of water producing normal consistency in the neat cement,  $s$  is the number of parts sand and  $k$  is a constant depending upon the character of the sand. In computing Table I of the specifications the value used for  $k$  was 6.5. This formula is empirical and was determined by averaging the consistencies which skilled operators assigned to a large number of different mortars.

#### DETERMINATION OF SOUNDNESS

##### 419. A. S. T. M. Method.\*

**42. Apparatus.**—A steam apparatus, which can be maintained at a temperature between 98° and 100° C., or one similar to that shown in Fig. 7, is recommended. The capacity of this apparatus may be increased by using a rack for holding the pats in a vertical or inclined position.

**43. Method.**—A pat from cement paste of normal consistency about 3 in. in diameter,  $\frac{1}{4}$  in. thick at the center, and tapering to a thin edge, shall be made on clean glass plates about 4 in. square, and stored in moist air for 24 hours. In molding the pat, the cement paste shall first be flattened on the glass and the pat then formed by drawing the trowel from the outer edge toward the center.

**44.** The pat shall then be placed in an atmosphere of steam at a temperature between 98° and 100° C. upon a suitable support 1 in. above boiling water for 5 hours.

**45.** Should the pat leave the plate, distortion may be detected best with a straight-edge applied to the surface which was in contact with the plate.

\* Unsoundness is usually manifested by change in volume which causes distortion, cracking, checking or disintegration.

Pats improperly made or exposed to drying may develop what are known as shrinkage cracks within the first twenty-four hours and are not an indication of unsoundness. These conditions are illustrated in Fig. 8.

The failure of the pats to remain on the glass or the cracking of the glass to which the pats are attached does not necessarily indicate unsoundness.

**420. Hints on Manipulation.**—Since the soundness test is the most important of the standard cement tests, great care should be exercised in

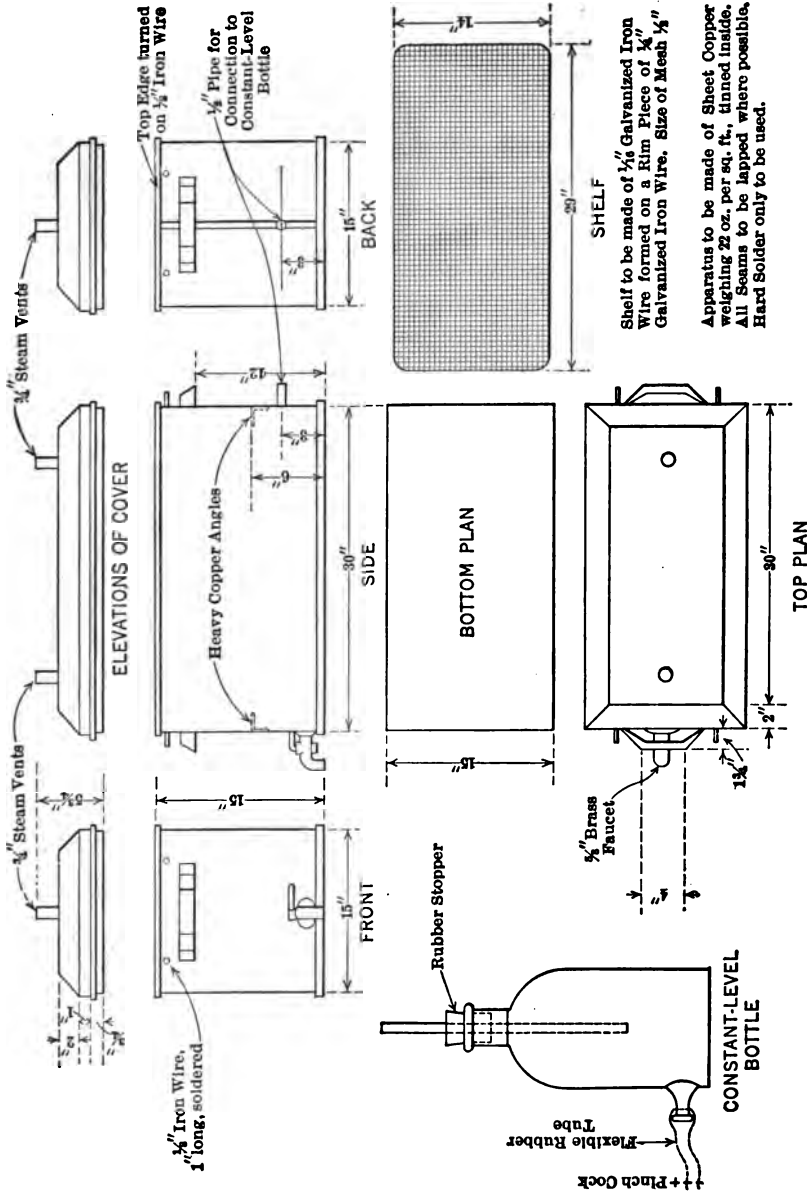


Fig. 7.—Apparatus for Making Soundness Test of Cement.

the fabrication and storage of the pats in order that the results shall not be vitiated by errors in manipulation.

Speed in roughly molding the pats may be gained by rapidly tapping the glass plate supporting the paste on the bench. This action quickly

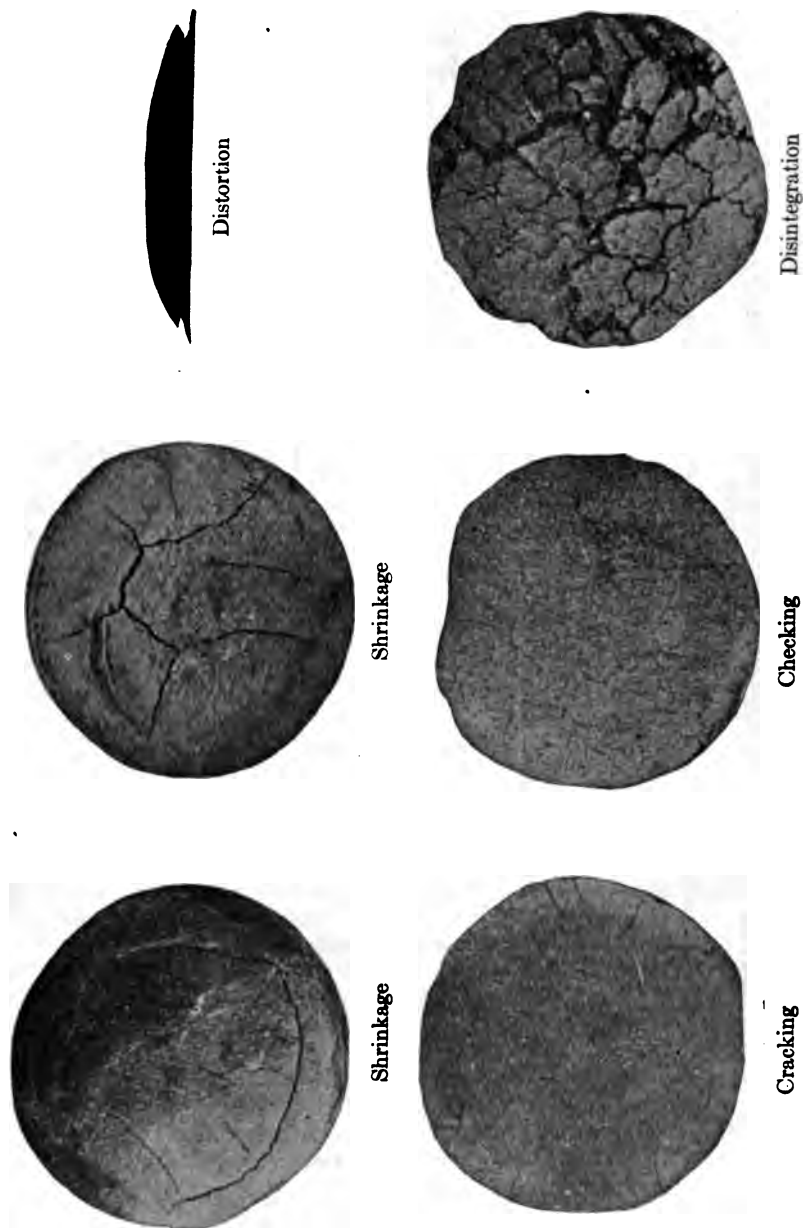


FIG. 8.—Typical Failures in Soundness Test.

spreads the paste into a flat circular pile and at the same time expels the air entrapped between the plate and pat. A common fault with the



unexperienced operator is to make the pat too wet or to allow it to dry by standing on the bench. Either of these errors is liable to cause shrinkage cracks. In marking the pats, deep abrasions of the surface should be avoided for similar reason.

**421. Le Chatelier's Test for Soundness.**—Several other accelerated tests for soundness have been devised. One of these which was recommended by the Committee on Accelerated Tests of the International Association for Testing Materials \* and specified in both the British and French Standard Specifications is the rough measurement of the expansion of a cylindrical test piece by Le Chatelier's tongs shown in Fig. 9. The method of making the test, which was originally proposed by Le Chatelier and adopted by the committee, follows: "The cement is gauged and filled into a mold on a plate of glass, the edges of the mold being held together. When the mold has been filled it is covered with a

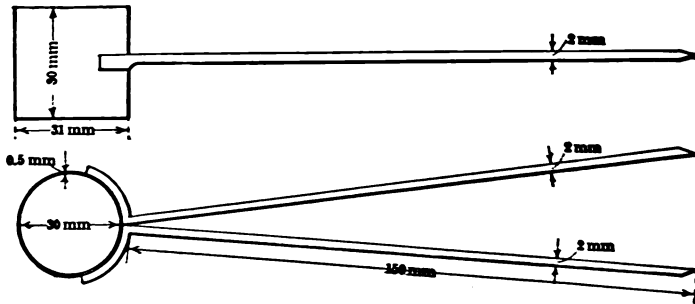


FIG. 9.—Le Chatelier's Tongs for Testing Constancy of Volume of Cement Pastes.

plate of glass held down by a small weight and the whole is immersed in water at  $15^{\circ}\text{C}$ . for twenty-four hours. Any tie or band which has been used to keep the edges of the mold together during setting is then removed. The distance between the indicator needles is measured and the mold is placed in cold water which is raised to a temperature of  $100^{\circ}\text{C}$ . in the course of half an hour and is kept boiling for six hours. The mold is removed from the water and after it has cooled the distance between the indicator needles is again measured. The difference between the two measurements represents the expansion of the cement. This must not exceed 10 mm. when the cement has been aerated for twenty-four hours and 5 mm. when the cement has been aerated for seven days."

**422. The Boiling Test.**—The boiling test recommended in 1870 by Dr. Michaelis has been considerably used for detecting unsoundness. In making this test a 2-in. ball of neat cement paste of standard consistency is allowed to harden in the moist closet for twenty-four hours, or

\* See *Proc. I.A.T.M.* 5th Congress, 1909 10; also *Engr. News*, Vol. 68, p. 506.

longer if hard set has not been obtained. It is then placed in a pan of pure water at normal room temperature. The temperature of the water is gradually raised to the boiling-point in not less than thirty minutes. After boiling for three hours the ball is removed and allowed to cool without sudden chilling. It is then examined for signs of disintegration.

**423. The Autoclave Test.**—Recently H. J. Force of the Delaware, Lackawanna & Western R.R. has devised a still more stringent accelerated test \* which has been inserted in the specifications for cement adopted by the above railroad. Inasmuch as the value of this test is a live issue at the present time a brief description of it is given. The test is a modification of the steam-pressure test of Erdmenger and is made in an autoclave—a metallic receptacle provided with a tightly fitting cover, pressure gage and safety valve. The method adopted by Force follows: Neat briquettes are molded and cured in a moist closet for one day in accordance with the A.S.T.M. standard method. They are then immersed in water in the autoclave which is gradually heated until at the end of three-quarters of an hour the pressure reaches 20 atmospheres. The pressure is then maintained constant for an additional  $1\frac{1}{4}$  hour after which the briquettes are removed and tested. Those pieces which come out of the autoclave in perfectly sound condition pass the test. Mr. Force claims, however, that briquettes subjected to this test ought to have 25 per cent greater strength than the normal neat briquettes tested at twenty-four hours.

**424. The Value of the Soundness Test.**—The standard accelerated soundness test is useful in detecting unsoundness caused by the hydration of finely ground free lime. It must be confessed, however, that some cements which fail in this test make sound concrete.

If a cement contains free lime which is coarsely ground and highly burned, it is doubtful if the standard test will always reveal the unsoundness †: it is contended, however, that the autoclave test will. Extensive experiments have been made by the Bureau of Standards (*Technologic Paper* No. 47) on the cements which passed and failed in the autoclave test and on concretes made from them. These experiments fail to show that the autoclave test is superior to the standard accelerated test as a measure of the soundness of concrete. It seems likely, however, that the autoclave test is a valuable index of the quality of cement used in making concrete products which are to be cured under steam pressure.

From the results of Prof. E. D. Campbell,‡ it appears that the stand-

\* See *Engr. News*, Vol. 67, p. 1111 and p. 1136; Vol. 68, p. 80; Vol. 69, p. 185; Vol. 70, pp. 40, 82, 277; *Proc. A. S. T. M.*, Vol. 14, p. 246. Also *Technologic Paper* No. 47, U. S. Bureau of Standards.

† See *Technologic Paper* No. 43, p. 58.

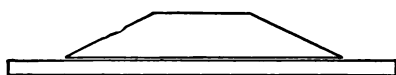
‡ *Journal of Industrial and Engineering Chemistry*, Vol. 8, p. 1101; also Art. 332.

ard steam pat test does not always detect free magnesia. His tests show that only 4 per cent of free magnesia is needed to cause water-cured cement prisms to expand linearly over 2 per cent. Fine grinding and proper burning of the raw materials will result in the production of sound cements when the raw materials run high in magnesia, but there appears to be no quick way to detect unsoundness in high magnesia cements.

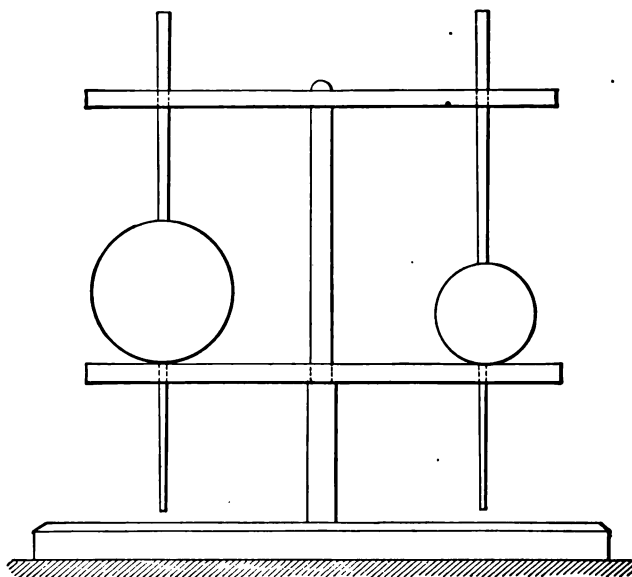
#### DETERMINATION OF TIME OF SETTING

##### 425. A. S. T. M. Method.

46. The following are alternate methods, either of which may be used as ordered:



(a) Pat with Top Surface Flattened for Determining Time of Setting



(b) Gillmore Needles

FIG. 10.

47. Vicat Apparatus.—The time of setting shall be determined with the Vicat apparatus described in Section 39. (See Fig. 6.)

48. Vicat Method.—A paste of normal consistency shall be molded in the hard-rubber ring *G* as described in Section 40, and placed under the rod *B*, the smaller end of which shall then be carefully brought in contact with the surface of the paste, and the rod quickly released. The initial set shall be said to have occurred when the needle ceases to pass a point 5 mm. above the glass plate in  $\frac{1}{2}$  minute after being

released; and the final set, when the needle does not sink visibly into the paste. The test pieces shall be kept in moist air during the test. This may be accomplished by placing them on a rack over water contained in a pan and covered by a damp cloth, kept from contact with them by means of a wire screen; or they may be stored in a moist closet. Care shall be taken to keep the needle clean, as the collection of cement on the sides of the needle retards the penetration, while cement on the point may increase the penetration. The time of setting is affected not only by the percentage and temperature of the water used and the amount of kneading the paste receives, but by the temperature and humidity of the air, and its determination is therefore only approximate.

**49. Gillmore Apparatus.**—The time of setting shall be determined by the Gillmore needles. The Gillmore needles should preferably be mounted as shown in Fig. 10 (b).

**50. Gillmore Method.**—The time of setting shall be determined as follows: A pat of neat cement paste about 3 in. in diameter and  $\frac{1}{2}$  in. in thickness with a flat top (Fig. 10 (a)), mixed to a normal consistency, shall be kept in moist air at a temperature maintained as nearly as practicable at 21° C. (70° F.). The cement shall be considered to have acquired its initial set when the pat will bear, without appreciable indentation, the Gillmore needle  $\frac{1}{16}$  in. in diameter, loaded to weigh  $\frac{1}{2}$  lb. The final set has been acquired when the pat will bear without appreciable indentation, the Gillmore needle  $\frac{1}{8}$  in. in diameter, loaded to weight 1 lb. In making the test, the needles shall be held in a vertical position, and applied lightly to the surface of the pat.

**426. Suggested Precautions.**—Too great emphasis cannot be laid upon the latter part of Section 48. If reliable results are to be obtained the specimens should be made in a room at approximately 70° F. The water used in mixing and the moist closet in which test-pieces should be cured ought also to be kept at the same temperature.

In using the Vicat apparatus, trial tests on time of initial set should be made near the circumference of the ring leaving the center portion unmarked for the final test. The needle should never be allowed to strike forcibly upon the plate *H*. Inasmuch as it is difficult to secure a smooth top surface without violating the recommendation in Section 40 some operators invert the specimen in determining final set.

**427. Comparison of Vicat and Gillmore Methods.**—It will be appreciated that the determinations afforded by either of the above methods are purely empirical and that there is no reason why the results obtained by one method should agree with those gotten by the other. The only conclusion which can be stated is that either time of set determined by the Vicat apparatus is less than that gotten from the Gillmore needles. The discrepancy may range from a few minutes to over an hour. In a series of tests involving 594 specimens, six cements and thirty-four laboratories (see *Rept. of Joint Conference on Uniform Methods and Standard Specifications for Cement* (April 28, 1915), the discrepancies averaged about 20 per cent for either initial or final set. For a comparison of results see Art. 352.\*

\* Several devices have been invented for automatically recording the setting of cement; see *Engr. News*, Vol. 46, p. 95; *Cement Age*, Vol. 13, p. 138, and Vol. 14, p. 88.

## TENSION TESTS

## 428. A. S. T. M. Method.

51. **Form of Test Piece.**—The form of test piece shown in Fig. 11 shall be used. The molds shall be made of non-corroding metal and have sufficient material in the

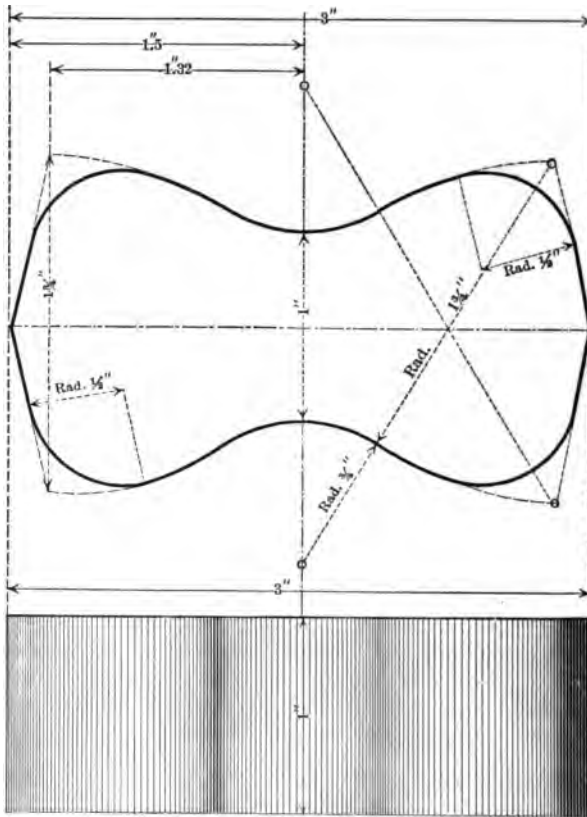


FIG. 11.—Details for Briquette.

sides to prevent spreading during molding. Gang molds when used shall be of the type shown in Fig. 12. Molds shall be wiped with an oily cloth before using.

52. **Standard Sand.**—The sand to be used shall be natural sand from Ottawa, Ill., screened to pass a No. 20 sieve and retained on a No. 30 sieve. This sand may be obtained from the Ottawa Silica Co., at a cost of two cents per pound, f. o. b. cars, Ottawa, Ill.

53. This sand having passed the No. 20 sieve shall be considered standard when not more than 5 g. pass the No. 30 sieve after one minute continuous sieving of a 500-g. sample.

54. The sieves shall conform to the following specifications:

The No. 20 sieve shall have between 19.5 and 20.5 wires per whole inch of the warp wires and between 19 and 21 wires per whole inch of the shoot wires. The diameter of the wire should be 0.0165 in., and the average diameter shall not be outside the limits of 0.0160 and 0.0170 in.

The No. 30 sieve shall have between 29.5 and 30.5 wires per whole inch of the warp wires and between 28.5 and 31.5 wires per whole inch of the shoot wires. The diameter of the wire should be 0.0110 in. and the average diameter shall not be outside the limits 0.0105 to 0.0115 in.

55. **Molding.**—Immediately after mixing, the standard mortar shall be placed in the molds, pressed in firmly with the thumbs and smoothed off with a trowel without ramming. Additional mortar shall be heaped above the mold and smoothed off with a trowel; the trowel shall be drawn over the mold in such a manner as to exert a moderate pressure on the material. The mold shall then be turned over and the operation of heaping, thumbing and smoothing off repeated.

56. **Testing.**—Tests shall be made with any standard machine. The briquettes shall be tested as soon as they are removed from the water. The bearing surfaces of the clips and briquettes shall be free from grains of sand or dirt. The briquettes

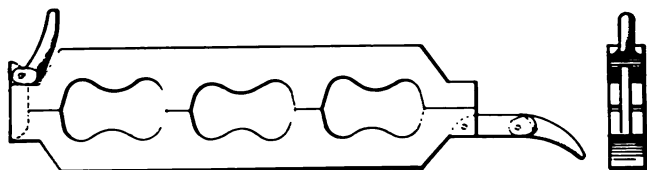


FIG. 12.—Gang Mold.

shall be carefully centered and the load applied continuously at the rate of 600 lb. per minute.

57. Testing machines should be frequently calibrated in order to determine their accuracy.

58. **Faulty Briquettes.**—Briquettes that are manifestly faulty, or which give strengths differing more than 15 per cent from the average value of all test pieces made from the same sample and broken at the same period, shall not be considered in determining the tensile strength.

**429. Reasons for the Tension Test.**—Although little reliance is placed upon the low tensile strength of concrete in making structural designs, yet the majority of strength tests on cements are tensile. This anomalous condition is due chiefly to three causes: first, the tensile strength is supposed to be a measure of the compressive strength; second, some authorities believe that the tensile strength affords quicker indications of defects in the cement than other strength tests; third, this test is more conveniently made than the compression test since the small specimens used may be tested on an inexpensive machine of small capacity. The first of the above arguments is decidedly fallacious, since the ratio of the strength in tension to the strength in compression varies considerably with the age of the test-piece, richness of mix, and the character of the cement. It may also be argued with considerable force that the form of test-piece,

methods of mixing and molding, and the method of gripping are of such nature that it is very difficult for different laboratories working under the present specifications to check results.

Experiments indicate that the results of compression tests do not vary more than tension tests of like mixes. Although a slightly more powerful testing apparatus is required for the compression test the added value of the results seems to justify the adoption of the compression test as the standard for measurement of strength. It is to be hoped that the specifications for standard compression tests on cement mortar, which have been tentatively advanced by the A.S.T.M.\* will, after whatever necessary modification is made, be adopted. In this connection it is of interest to note that the German Government,† after requiring the tension test for many years, has discarded it and now retains a requirement for the compressive strength of 1 : 3 mortar.

**430. Indications Afforded by Neat and Mortar Tension Tests.—**Although neat cement paste, both on account of high cost and unstability under temperature and atmospheric changes, is not used in construction; yet it has been customary to make tests on both neat and 1 : 3 mortar specimens. Owing to the fact that neat tensile tests afford little information beyond that gotten from time of set and soundness tests, they were abandoned when the specifications were revised in 1916.

Tension tests of mortar briquettes are valuable in determining the quality of the cement if always made under identical conditions. They also afford a very rough index of the quality of the concrete which can be made with the cement.

When the time allowed for testing is short, tests on 1 : 3 mortar briquettes of standard sand give valuable indications of quality. Such briquettes should exhibit a strength of 150 lb. per square inch. Care must be taken, however, to maintain moist closet and water bath at 70° F., since tests at the University of Wisconsin have shown that the strength of 3-day mortar briquettes may be decreased 20 lb. and that of 7-day briquettes 10 lb. by immersion in water at 60° F.

**431. The Theory of the Distribution of Stress over the Minimum Section of a Cement Briquette** has been developed by M. Durand-Claye in *Annales des Ponts et Chaussées*, June, 1895. He found for the form of briquette shown in Fig. 11 that the ratio of the maximum to the minimum unit stress was 2.12 to 1 and the ratio of the maximum to the average unit stress was 1.54 to 1. Prof. Coker in *Proc. I.A.T.M.*, 6th Congress, 1912, gave proof based on experiment that the latter ratio was 1.7 to 1. These investigations show that the intensity of stress is greatest along the sides of the minimum section and least at the center.

\* See Report of Committee C-1, *Proc. A.S.T.M.*, 1916.

† See *Engr. News*, Vol. 64, p. 214.

**432. Precautions to Observe in Molding Briquettes.**—The operator should note well that Section 55 of the Specifications prohibits the use of rammers in filling the molds. When such devices are used, or when the molds are filled in thin layers, the early strength of the specimens will be greater than if made in the standard manner. Care should also be taken to see that the minimum cross-section of the briquette is exactly 1 in. thick. Briquettes varying in thickness from the above dimension will be

eccentrically loaded when tested in standard grips; and, as a result, their load-carrying capacity will be decreased.

**433. Mechanical Mixer.**

—Although it is doubtful if mechanical mixing and molding devices will ever be favored by American engineers for routine testing, yet their value is evidenced by the fact that such apparatus is prescribed in the German Standard Specifications. For research work such machines may be found quite convenient. The Steinbruch-Schmelzer mixer which is adopted in the above specifications is shown in Fig. 13. Before the mortar is introduced into this machine, it is customary to

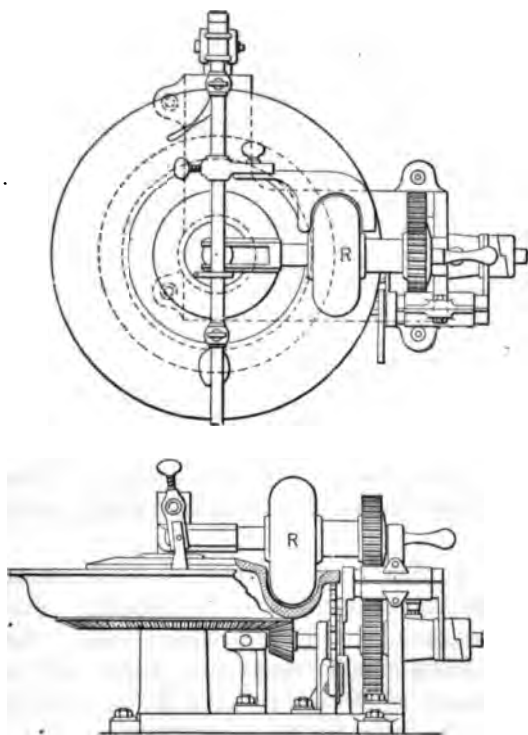


FIG. 13. — A Steinbruch-Schmelzer Mortar Mixer.

The roller (*R*) weighs with its axle about 48 lb. It is 3.15 in. thick and 7.9 in. in diameter. (*Engr. Record*, Vol. 61, p. 819.)

mix the materials in a bowl with a spoon. In the mixer the mortar is carried under the heavy roller (*R*) by the trough which whirls about a vertical axis in the same direction as the roller but at a faster rate. Two scrapers attached to the axis about which the pan revolved are also provided to work and turn over the mortar.

**434. Böhme Hammer.**—The molding machine which has been commonly employed in Germany and to a slight extent in the United States is the Böhme hammer, a modification of which is shown in Fig. 14. The 4½-lb. steel hammer of this apparatus is driven by a wheel provided with



ten cams which is geared to a crank and handle. An automatic stop is provided which acts after 150 blows, this being the number usually given each specimen. The apparatus may be employed to make prisms and cubes as well as briquettes.

**435. Types of Testing Machines.**—A very efficient and simple device for testing the strength of briquettes is shown in Fig. 15. In starting the test, sufficient fine shot is placed in the bucket, *b*, to counterbalance the weight, *w*. After the briquette is adjusted in the grips or clamps, *g*, the trigger, *t*, at the top of the bucket is released; this opens a piston valve which allows shot to flow down into the cup, *c*. As the shot flows from the bucket the weight, *w*, moves downward; and, through the lever

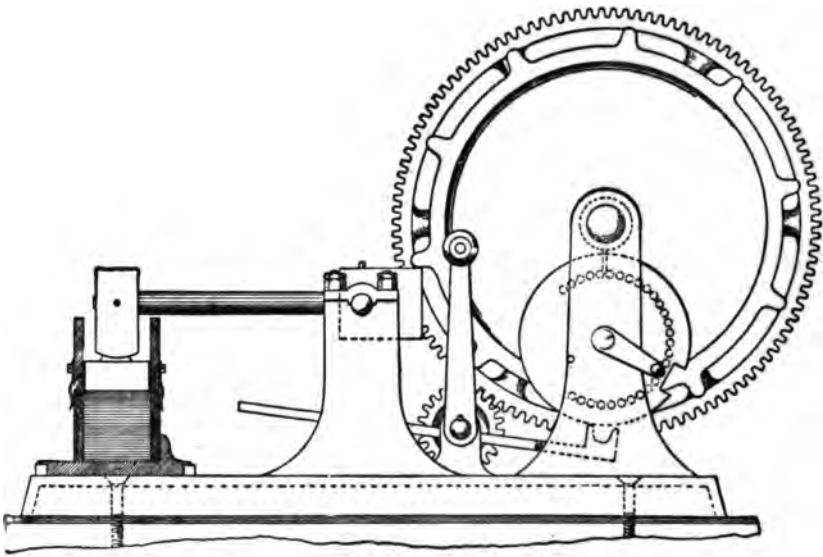


FIG. 14.—Böhme's Hammer for Making Cement Briquettes.

system, a strain is produced upon the briquette which is proportional to the shot lost. To keep the beam in the central position, the crank, *k*, is turned in the clockwise direction. When the briquette breaks, the bucket moves upward but the piston is held by the rod, *r*, so that the valve is immediately closed and the flow of shot stopped. The spring balance is graduated to read the load on the briquette to the nearest 5 lb.

Another type of machine in which the load may be applied by hand is shown in Fig. 16. In this machine the poise is driven at a constant speed by means of the small motor shown at the right. The operator keeps the needle beam balanced by applying load with the horizontal hand wheel, which is placed above the bed of the machine and near the motor.

**436. Necessity of Using Roller Clips.**—Experiments made at the University of Wisconsin on 135 neat briquettes show that variations in strength of 8 per cent due to differences in roller clips is quite possible. Larger discrepancies were encountered with grips which had no roller bearings. The tests indicate apparently that the rollers should have a bearing over the entire width of the briquette and should be lubricated so that they will turn easily.

**437. The Effect of Eccentric Loading on the Strength of Briquettes.**—In making briquette tests it is of great importance that the load shall be

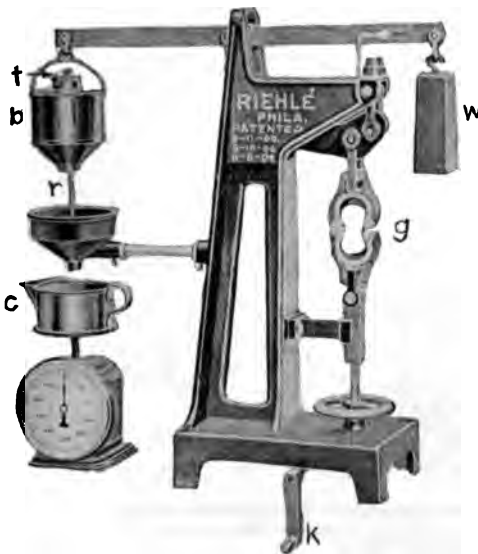


FIG. 15.—An Automatic Briquette-testing Machine. (Capacities 1000 or 2000 lb.)



FIG. 16.—A Motor-driven Briquette-testing Machine of 2000 lb. Capacity.

centrally applied. The operator should, therefore, see that any projecting edges are removed from the specimens, that the rollers work freely, that the specimen rests against the bars on the back of the clips and that the clips are perfectly aligned upon the test-piece. Experiments made at the Massachusetts Institute of Technology \* show that a displacement of  $\frac{1}{16}$  in. will decrease the tensile strength from 15 to 20 per cent.

Since briquettes become more brittle with age the effect of a slight eccentricity or any torsional strain will be more marked in long-time tests than in those made at an early age.

\* *Trans. Am. Soc. Mech. Engr.*, Vol. 9, p. 181.

**438. The Effect of the Rate of Loading on the Strength of Briquettes.—**

A number of series of experiments have been made to determine the effect of rate of loading on the tensile strength of briquettes. These show that the strength increases with the rapidity of loading. One should be careful, therefore, to keep the rate of loading within 100 lb. per minute of that specified in order that errors due to this source may be less than 2 per cent.\*

**439. Number of Specimens.—**It is good practice to break from three to five briquettes at each period. If the work is carefully done an individual result should not vary more than 10 per cent from the mean.

**STORAGE OF TEST PIECES****440. A. S. T. M. Method.**

**59. Apparatus.—**The moist closet may consist of a soapstone, slate or concrete box, or a wooden box lined with metal. If a wooden box is used, the interior should be

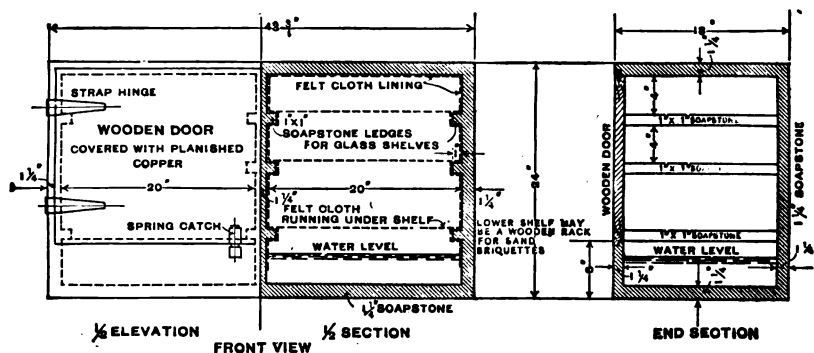


FIG. 17.—Moist Closet. (From *Concrete, Plain and Reinforced*; by Taylor & Thompson.)

covered with felt or broad wicking kept wet. The bottom of the moist closet should be covered with water. The interior of the closet shall be provided with non-absorbent shelves on which to place the test pieces, the shelves being so arranged that they may be withdrawn readily.

**60. Methods.—**Unless otherwise specified all test pieces, immediately after molding, shall be placed in the moist closet for from 20 to 24 hours.

**61.** The briquettes shall be kept in molds on glass plates in the moist closet for at least 20 hours. After 24 hours in moist air the briquettes shall be immersed in clean water in storage tanks of non-corroding material.

**62.** The air and water shall be maintained as nearly as practicable at a temperature of 21° C. (70° F.).

**441. Moist Closet.—**Inasmuch as the humidity of the atmosphere in which the specimens are stored has an important effect upon setting and strength, pains should be taken to maintain complete saturation of the air in the moist closet. A suitable form of closet is that shown in

\* See experiments by W. P. Taylor in his *Practical Cement Testing*, p. 148; also Wheeler's tests, *Report of Chief Engrs., U.S.A., 1895*, p. 2951.

Fig. 17; one made of concrete is described by E. B. McCready in the *Engr. News*, Vol. 58, p. 107.

**442. Storage Bath.**—After specimens are removed from the moist closet they should be stamped or marked with a lead pencil or water-proof crayon before immersion in the water bath. Briquettes should be marked near the ends. The bath should be supplied with pure running water or the water in the tank should be changed daily. If this is not done, the salts liberated in curing form solutions which affect the strength of the specimens. A convenient water bath is shown in Fig. 18.

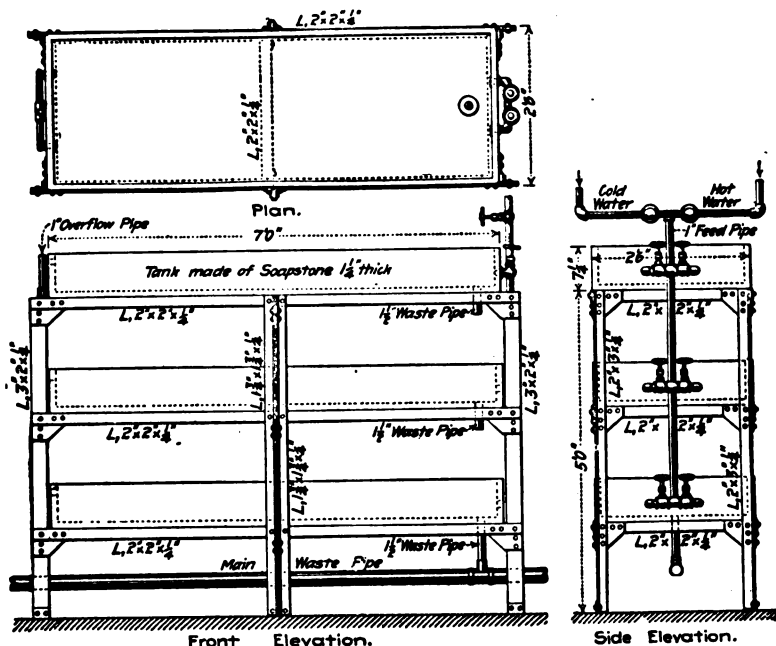


FIG. 18.—Sketch of Storage Tanks for Briquettes. (From Taylor's *Practical Cement Testing*.)

For the effect on strength of early removal of specimens from storage bath see Fig. 11, Ch. IX.

#### THE INTERPRETATION OF THE RESULTS OF STANDARD TESTS

**443. General Recommendations.**—The student should always bear in mind that the tests are only of qualitative value for the purpose of comparing the sample with a standard, adopted after long experience, or to compare it with previous samples of the same brand of cement. Therefore, before accepting or rejecting a cement, he should carefully consider the relative value of the results of each test and the conditions

under which the cement is to be used. Furthermore, one should not condemn a sample unless certain that the conditions surrounding each test were standard. Any uncertainty in regard to a result should be removed by performing a second test under standard conditions. Below will be found a brief discussion of the criteria determining the acceptability of Portland cement arranged in order of the importance of the various tests.

**444. Soundness.**—If one or more steamed pats show signs of distortion, checking, disintegration or radial cracks (Fig. 8) the cement should be rejected; or if conditions permit the cement may be stored and further soundness tests made (Sec. 14 of Specifications).

Although the accelerated soundness tests is made under abnormal conditions, yet the results of a thousand boiling tests made by W. P. Taylor on different brands of cement bear witness to its value. He found that about one-third of the cements failing in the boiling test gave evidence of unsoundness in air-cured pats or a retrogression in strength inside of twenty-eight days; within one year approximately seven-eighths of these cements had given like evidence of unfitness. Furthermore, only one-half of 1 per cent of the cements which passed the boiling test showed signs of unsoundness in the air pat tests and only 13 per cent retrograded in strength within the year.

**445. Tensile Strength.**—Reject the cement if the strength at twenty-eight days is less than required by the specifications. If the strength of the mortar briquettes at twenty-eight days is less than at seven days, reject. If the strength of mortar briquettes is a trifle below the standard at seven days but is above at twenty-eight days, accept the cement. If judgment must be passed upon a sample at the end of seven days, reject on a decidedly low mortar test. Hold for twenty-eight days if the mortar strength is slightly below the standard.

**446. Time of Set.**—If the cement does not pass the specification in the laboratory test, determine its behavior under conditions in which it will be used before condemning it. Slow final set many times accompanies coarseness, which will also affect the tensile strength of the mortar briquettes.

**447. Fineness.**—Inasmuch as no sieves made at the present time are sufficiently fine to determine the percentage of flour, the important cementing element, this test has only corroborative value. Generally, a coarse cement will exhibit a low mortar strength and will oftentimes fail in the soundness test. Cements failing in the fineness test should not be accepted before the twenty-eight day tests are made.

**448. Specific Gravity.**—Like the fineness test, the specific gravity is oftentimes of corroborative value. Its chief value is to detect adulteration when the average specific gravity is known. Such adulteration

may affect the mortar tensile strength and cause unsoundness. The importance of this test is small.

For further information on interpretation of results of cement tests, consult W. P. Taylor's *Practical Cement Testing* or R. K. Meade's *Portland Cement*.

#### MISCELLANEOUS METHODS OF TESTING CEMENTS

**449. Methods of Making Cross-bending Tests of Cement.**—Cross-bending tests of cement have been advocated occasionally, but they have not come into general use anywhere. The French Commission have also undertaken to standardize this test. They recommend a specimen 5 in. (120 mm.) long and 0.8 in. (20 mm.) square in cross-section.

M. Durand-Claye has shown by very extended series of tests in tension and in cross-bending, on identical samples of neat Portland cement, that the average ratio of the modulus of rupture in cross-bending to the tensile strength, as determined upon standard forms of briquettes, is 1.92 at seven days and 1.86 for twenty-eight days, or an average of 1.89.\* This relation was found to subsist between averages made up from the means of the three tests in each set of six, in both tension and cross-bending. The mean error of a single test at twenty-eight days was found to be 2.10 per cent for the tension tests and 2.13 per cent for the tests in cross-bending, thus showing that the two methods of testing were equally accordant.

It would seem, therefore, that tests in cross-bending may be employed with assurance as a means of determining both relative and absolute values of cements and cement-mortars, their principal disadvantage lying in the fact that there are few records extant with which to compare the results of such tests.

The principal recommendation for the use of transverse tests would seem to lie in the economy of a testing outfit. It has been estimated that a suitable machine for testing cement transversely could be constructed for about \$12, while a set of molds for sixteen prisms would cost not to exceed \$3, or if these latter be made of cast iron the cost need not exceed \$5 per set of twelve after the patterns are made.†

It is further claimed that since all transverse breaks are fair, while with the forms of briquettes and clips in use in America a considerable percentage of the breaks occur outside of the minimum section, the results of transverse tests must be more reliable. Nevertheless, there seems to be now in this country no inclination to change from tension to transverse tests of cement.

\* Messrs. Abbott and Morrison, in their thesis published in *Engineering News*, Dec. 14, 1893, show that for neat cement this ratio was 1.8 on prisms 1 in. square and broken on a span of 4 in.

† See *Engineering News*, Vol. 30, p. 469, where complete detail drawings are given of both the machine and of the molds.

**450. Methods of Testing the Adhesion of Cement and Cement Mortars to Various Substances.**—While the tensile strength of briquettes shows the cohesion of the mortar, it has been found by experiment that its adhesion either to other mortars or to the same mixture which has already hardened, or to brick or stone or metal, is very much less than its cohesion. It is important, therefore, to have a standard test of adhesion, as well as of strength. Because tests of this kind are comparatively new, no general custom has been established in America on the subject; but the following recommendations have been made by the French Commission:

(1) For tests of adhesion of cements and cement-mortars use will be made of a special form of briquette, molded in two parts, these two parts consisting of the two materials whose adhesion is to be tested, provided both can be molded, or containing between them a prism of the solid body to which the adhesion of the mortar is to be determined. The form of this briquette, as modified for English units, with 1 sq. in. of area on the surface of adhesion, is shown in Fig. 19. This mold is formed in two parts, and is used to form in succession the two halves of the complete briquette.

(2) *To compare the force of adhesion of different cements to a given material, normal adhesion-blocks* will be prepared as follows: Use for these always one kind of standard Portland cement which has passed a sieve of eighty meshes to the linear inch, mixed with the standard sand No. 3 (sand passing a No. 15 and held on a No. 11 sieve) in the proportion of one of cement to two of sand. These normal adhesion-blocks will be molded in the form of one-half of the briquette shown in Fig. 19.\* It will be gauged with 9 per cent of water and rammed into the mold. At the end of twenty-four hours in air it will be placed in fresh water for a period of at least twenty-eight days. When it is to be used, it will first be dried and its adhesion-surface polished with emery-paper.

(3) The cement to be tested for adhesion with these standard blocks prepared as above will be mixed as a normal plastic mortar, one of cement to three of sand (the consistency advocated is about the same as the

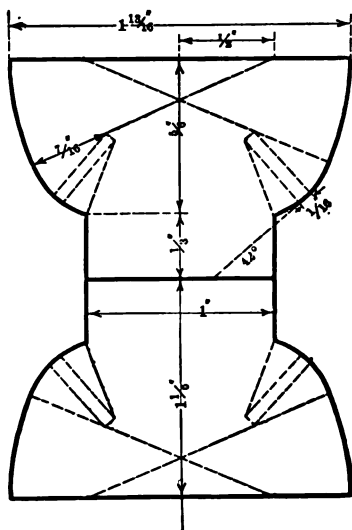


FIG. 19.—Form of Briquette for Adhesion Test of Cement as Adopted by the French Commission and Adapted to English Units.

\* The detail drawings of these molds are given in *Com. des Méthodes D'Essai Mat. Const.*, Vol. 4, p. 284.

forty-eight hours. It should then be immersed in cold water which is raised to the boiling-point and maintained at that temperature for two hours. Without removing the block it should be allowed to cool and then weighed after twenty-four hours.

#### 453. Methods of Testing the Permeability of Cements and Mortars.—

The permeability of cement-mortar is often a very important matter, as in the case of reservoir wells and linings, and often in foundation walls placed below the level of the ground-water. Neat cement-mortar is absolutely impervious when it has hardened and has not cracked, and so also is a mixture of one to one, or even of two of sand to one of cement, by weight, if well mixed. The normal mixture of three of sand to one of cement may also be made practically impervious with the most thorough mixing of the dry ingredients and a compacting of the mortar by hard ramming.

Professor Tetmajer has used the apparatus shown in Fig. 20 to obtain a modulus of permeability. Here a cylinder of the mortar is made and allowed to harden under water for a specified time. It is then mounted in the apparatus by means of annular rubber-cushion or packing discs, and the water let on below under a known pressure. The permeability of the mortar is indicated by the rate at which the water passes the disc and rises in the glass tube above, which is graduated to cubic centimeters. J. B. Johnson also used this apparatus with satisfactory results, a convenient pressure to use being that of the city water-mains.\*

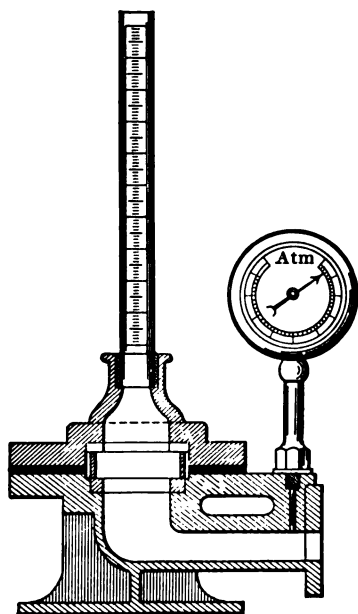


FIG. 20.—Tetmajer's Apparatus for Testing the Permeability of Cement-mortar. (Mitteilungen der Material - prüfungs - Anstalt, Poly. Zurich, Vol. 6.)

The French Commission recommended a standard permeability test as follows:

(1) The permeability of cement-mortars will be indicated by the number of liters of water passing per hour through a cubical block of 7 cm. (say  $2\frac{3}{4}$  in.) on a side, under the following conditions.

The water will be brought to the top face of the specimen, laid edge-wise (what was the horizontal plane in the formation of the cube now

\* A modification of this method has been used by the U. S. Bureau of Standards, see Art. 522.



becoming a vertical plane), through a glass tube, 35 mm. internal diameter and about 4 or 5 in. high, which is sealed to the top face of the cube by neat cement-mortar as shown in Fig. 21. A rubber tube connects the upper end of the glass tube with the reservoir placed at a height (from the surface of the water of immersion to the surface of the water in the reservoir) of 4 in., 40 in., or 400 in. (0.1 m., 1.0 m., or 10.0 m.).

Before beginning the experiment the cube of mortar is to be immersed in water for forty-eight hours, and during the test the block is to remain immersed to prevent the formation of an impervious coating on the outside from the evaporation of the exuding water.

The volume of water passing will be given for the standard periods of twenty-four hours, seven days, twenty-eight days, and three months. For very porous mortar a shorter period than twenty-four hours may be employed, and at the same time the head of water must be stated.

Tests will be made on three similar specimens, the mean of the two most accordant results to be used.

(2) The *normal test* of permeability will be made on cubes made up of *normal plastic mortar* (3 sand to 1 cement, by weight) as described in Art. 416,\* and the specimen cubes must harden in water under the normal conditions for twenty-eight days before testing.

For tests on other mixtures, and for other periods of hardening, they recommend that mixtures of 2 sand to 1 cement, and 5 sand to 1 cement, by weight, and hardening periods of seven days, twenty-eight days, and three months be chosen.

In all cases the composition, age, and conditions of hardening must be stated, as well as the amount of water passed and the pressure-head used.

The method and form of specimen adopted at the University of Wisconsin for testing concrete (see Art. 524) can also be very effectively used in testing the permeability of cements and mortars.

\* The consistency referred to is practically the same as that recommended by the French Commission.

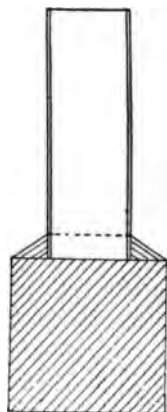


FIG. 21.—Apparatus for Testing the Permeability of Cement - mortar. (Recommended by the French Commission.)

## CHAPTER XIII

### MAKING MORTAR AND CONCRETE

**454. Introduction.**—When using concrete and mortar in construction the engineer is placed in the position of a manufacturer who fabricates a finished structural product from several raw materials. It is, therefore, necessary that he should be thoroughly cognizant of the properties of the constituent materials and the methods of handling them in order that a satisfactory product will result. In view of this consideration and in view of the great number of concrete constructions now being built, we shall pay considerable attention to the important factors which enter into the making of mortar and concrete.

It will be appreciated that very often the ideal conditions hereinafter mentioned cannot be attained in practice: Old cement must sometimes be used; only a fine-grained sand is attainable; the gravel is dirty or is of decidedly variable gradation and cannot be economically screened; a very wet consistency must be used on account of difficulties encountered in placing the concrete; or no care can be given to providing suitable curing conditions. Very often such undesirable conditions may be offset, to a large extent, by increasing the proportion of cement in the mixture. Many times it will be found economical to use local material of poor quality in a rich mixture rather than to ship in a superior sand or gravel and use a lean mix. Then, too, there is often a wide variation in the finished concrete due to methods of handling.\* Changes in consistency may easily produce variations of 100 per cent in strength and the methods of mixing and placing adopted by different concerns may also be responsible for as wide variations in properties. All of these facts emphasize the necessity of testing samples of concrete, made under the conditions of practice, before deciding upon the proper mixture to use; and also, they point out the importance of testing the product as used in the work.

Owing to the haste with which concrete structures are often built and the lack of facilities for testing, there has been a great reticence on the part of engineers to specify tests of concrete. They have framed specifications containing demands for certain more or less easily determined properties of the cement, sand, and broken stone; have called for certain proportions, and have stipulated a method of mixing; but definite specifications for the properties of the manufactured product are in most cases wanting.

\* See results obtained by U. S. Bureau of Standards in *Technologic Paper No. 58*.

An undue reliance is placed upon the acceptance tests of the constituent materials. These tests are valuable and serve to eliminate most poor material. Nevertheless, the acceptance tests sometimes advocated for sands are not always true criteria of the relative values of the concretes made from them, and sometimes cause the rejection of satisfactory material. Bearing these facts in mind, we shall now consider the properties of aggregates and the methods of testing and proportioning them. In order to appreciate better the significance of certain much-used terms their definitions will receive first attention.

### DEFINITIONS

**455. Mortar.**—A mixture of sand, screenings, or similar inert particles, with cement and water which has the capacity of hardening into a rocklike mass is called mortar. In general the maximum size of the inert particles in mortars is less than  $\frac{1}{4}$  inch.

**456. Concrete.**—A mixture of crushed stone, gravel, or similar inert material with a mortar is called concrete. The maximum size of inert particles in concrete is variable but is ordinarily under two inches. Rubble concrete is made by embedding a considerable proportion of large boulders or blocks of stone in concrete.

**457. Cement.**—The binding agencies most commonly employed in making concrete are the hydraulic cements and the bitumens. In this book only concrete and mortar made from the hydraulic cements will be considered; and, whenever, in this and succeeding chapters, the term concrete or mortar appears unqualified, Portland cement should be understood to be the binder. For information concerning the properties, the methods of testing and the standards for cements, reference should be made to Ch. IX, X, XI and XII.

**458. Aggregate.**—The inert particles within concrete or mortar comprise the aggregate. Although no hard and fast division can be made sands, screenings, mine tailings, pulverized slag, etc., in which the max-

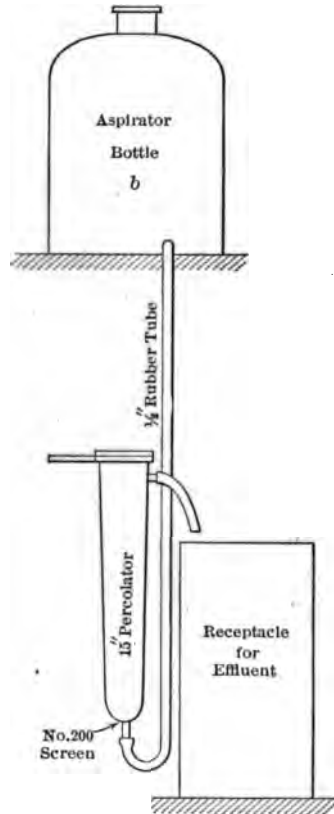


FIG. 1.—Apparatus for Determining Amount of Silt in Aggregate.

mum diameter of particles is less than  $\frac{1}{4}$  in. may be classed as fine aggregate; crushed stone, gravel, cinders, slag and the like containing larger particles are called coarse aggregate.

**459. Silt** is the impalpable dust which is present, at least in small proportion in nearly all aggregates. The amount of silt can be determined with the apparatus shown in Fig. 1. One or two hundred grams of dry fine aggregate is carefully weighed, placed in the percolator and washed by a stream of pure water from the bottle (b). The silt is continuously stirred with a glass rod until the effluent from the percolator becomes clear. The residue in the percolator is then dried, and the loss in weight



FIG. 2.—Voidmeter (a); Specific Weight Apparatus (b).

determined. If an analysis of the silt is desired the effluent from the percolator is filtered, and the residue on the filter is dried and analyzed. In order to obtain consistent results the dimensions of the percolator, velocity of flow of water, size of orifice and method of stirring should be standardized.

**460. Specific Weight.**—The weight of an aggregate contained in a measure of unit volume is called its specific weight. Generally the units of volume used in this country are the cubic foot and the cubic yard.

Inasmuch as this determination is affected by the moisture content of the aggregate due consideration of the latter should be given. In specific weight determinations on fine aggregate the results for dry material cannot be accurately computed from tests in which the moisture content is sufficient to wet the surfaces of the particles. (See Art. 471.) Obviously the degree of compactness of the material also greatly influences the specific weight; hence the adjectives rammed, compacted, shaken-down and loose are used.

Unfortunately no absolute standards of measurement have been agreed upon, but a suitable laboratory device for making determinations of the weight per cubic foot loose is shown in Fig. 2(a). By controlling the size of the opening between the trapdoors of this apparatus the aggregate in the hopper is allowed to fall 3 ft. at the rate of a cubic foot per minute and the surplus material is struck from the top of the cubic foot measure before the latter is moved to the scales. In the field a like degree of compactness may be gotten by allowing the aggregate to slide off a shovel held at a

height of 3 ft. above a 14-qt. pail. Loose measurements of volumes are commonly made, and unless otherwise specified this method will be understood herein.

**461. Voids.**—If an aggregate is poured into a container of any sort it will be observed that not all of the space within the container is filled. To the vacant spaces between the particles of aggregate the name voids is applied. Necessarily, the percentage of voids like the specific weight is affected by the compactness of the aggregate and the amount of moisture which it contains. Generally void determinations are made on material measured loose.

There are two classes of methods commonly employed for measuring voids, the direct and the indirect. The most-used direct method consists in determining the volume of liquid, generally water, which is required to fill the voids in a given quantity of material. Since in pouring water into fine aggregate it is impossible to expel all the air between the particles, the measured voids are smaller than the actual. It, therefore, becomes evident that the above direct method should not be used with fine aggregates unless the test is conducted in a vacuum. By the indirect method, the solid volume of a known quantity of aggregate is gotten by pouring the material into a calibrated tank partially filled with water; the difference between the apparent volume of material and the volume of water displaced equals the voids. If very accurate results are desired void measurements should be corrected for the porosity \* of the aggregate and the moisture it contains. If the latter corrections are made the specific gravity may also be gotten by this method (see Art. 408). Fig. 2(b) shows a convenient device for measuring voids by this indirect method.†

When the specific gravity of the aggregate is known the voids may be calculated thus:

$$\text{Per cent. voids} = \frac{62.5 \times \text{sp. gr. of aggregate} - \text{wt. per cu. ft. of aggregate}}{0.625 \times \text{sp. gr. of aggregate}}.$$

The variation in the size of particles in the aggregate greatly affects the voids. If a mass of spheres of equal diameters be piled as compactly as possible it may be shown that the per cent voids is approximately 26.‡ If spheres of smaller diameter, which will just fit into interstices in the original pile, are added, it is evident that the voids may be reduced. By the insertion of a third lot of still smaller spheres within the voids left in the pile a further reduction in voids may be made. Although particles of aggregate cannot be assigned to such definite positions, especially if they are loosely piled, yet experiments have shown that the voids in loose

\* See Art. 269.

† For other devices for determining voids see *Bulletin* No. 329, U. S. G. S.

‡ See *Concrete Plain and Reinforced*, by Taylor and Thompson, 3rd Ed., p. 130, for demonstration.

material may be greatly decreased by properly grading the sizes of the constituent particles.

**462. Mechanical analysis** consists in determining the proportionate amounts of particles held on or passing through a series of sieves differing in size of mesh. Such analyses indicate whether the material is properly graded to produce the minimum voids, when it is made into mortar or concrete. They also provide a means of studying how the gradation of particles may be improved by making proper additions and how different grades of aggregates may be combined with cement to form the most efficient mix.

For analyzing fine aggregate it is common practice in this country to employ sieves made of woven-brass wire. In some laboratories sheet brass perforated with circular holes is used instead of the wire cloth. The number of meshes per linear inch, the diameter of wire and size of opening in the most-used sieves are given in the upper part of Table 1. The sieves listed in the lower part of the table are sufficient for most gradation tests on fine aggregates. Sieves for fine aggregate are generally made 8 in. in diameter and 2¼ in. deep. They are so fashioned that they may be nested one above the other with the sizes arranged in order of fineness of mesh from the bottom upwards.

TABLE 1.—SIZES OF OPENINGS AND DIAMETER OF WIRES IN SIEVES FOR FINE AGGREGATE.

Mesh.	Size of Opening. (In.)	Diameter of Wire. (In.)	Mesh.	Size of Opening. (In.)	Diameter of Wire. (In.)	Mesh.	Size of Opening. (In.)	Diameter of Wire. (In.)
Sieves commonly used								
4*	.203	.047	20*	.034	.016	80	.0068	.00575
8*	.097	.028	30*	.022	.011	100*	.0055	.0045
10*	.077	.023	40	.014	.011	120	.0046	.0037
12	.060	.023	50*	.011	.009	150	.0041	.0026
16*	.049	.018	60	.0092	.0075	200*	.0029	.0021
A Satisfactory Series of Sieves.†								
4	.185	.065	28	.0232	.0125	200	.0029	.0021
8	.093	.032	48	.0116	.0092			
14	.046	.025	100	.0058	.0042			

\* Most-used sieves.  
† This is a portion of the standard screen scale manufactured by the W. S. Tyler Co., Cleveland. The intermediate sizes 6, 10, 20, 35, 65, and 150 are desirable equipment, but superfluous in many analyses. The main advantage of this sieve lies in the fact that the sizes of opening in the sieves vary in accordance with a constant ratio.

For coarse aggregate, sieves are made from woven-wire meshing sheet metal perforated with round holes. The size of opening in the clear, is stated in fractions of an inch. Wooden-rimmed sieves 16 to 20 in. in diameter and 4 in. deep equipped with meshing or

ated metal varying in diameter of opening from  $\frac{1}{8}$  in. to 1 in. by  $\frac{1}{8}$  in. intervals and from 1-in. to  $2\frac{1}{2}$ -in. by  $\frac{1}{4}$ -in. intervals form a convenient series for laboratory use. For ordinary use the following sizes are convenient: 2,  $1\frac{1}{4}$ ,  $\frac{3}{4}$ ,  $\frac{1}{2}$ ,  $\frac{1}{4}$  in.

Before using, all sieves should be carefully calibrated to ascertain the variation in shape and size of openings and the average diameter of the mesh. For fine sieves a linen tester's microscope may be used to count the meshes but a microscope fitted with a graduated eye-piece is necessary to measure accurately the size of the wire. Coarser sieves may be measured by means of micrometer calipers. The most accurate calibrations can be made by shaking materials of known analysis upon the given sieve and determining the per cent passing. The diameter of opening in a given sieve should not vary more than 5 per cent from the specified average diameter.

In making the mechanical analysis test a representative sample\* of the aggregate should be dried and weighed on scales accurate and sensitive to 0.1 per cent of the weight of the total sample. For fine aggregate a 1000-gram sample is a convenient quantity. Commonly, the sieves are nested in a mechanical shaker, a type of which is shown in Fig. 4, Ch. IX. The sample is poured on the top sieve, the cover secured in place, and the apparatus is run for a length of time which has been found sufficient to separate similar material. For determination on coarse aggregate, a 50- to 100-lb. sample, depending upon the range in size of particles, should be used. In the laboratory, this sieving is generally done by hand starting with the coarsest sieve.

When weighing the residues on the sieves, it is convenient to place the material passing the finest sieve in the scale pan first, then add the material passing the next coarser sieve and so on. By this procedure the successive weight readings may be easily converted into per cents passing and the discrepancy between the final and initial weight of the sample furnishes a check upon the losses in sieving.

Interpretation of the results of mechanical analyses are most readily made by means of diagrams. In these diagrams per cents by weight passing a given size of mesh are plotted as ordinates and the diameters of openings in the mesh as abscissas. Examples of such diagrams may be seen in Fig. 5.

**463. Yield.**—The ratio of the volume of mortar or concrete to the volume of aggregate contained in the mix is called the yield. The value of this index is apparent in computing quantities of materials required to fill a given set of forms with concrete or mortar. Since sand, gravel and broken stone are generally sold by loose measurement, it is customary to base the yield upon volume of loose material.

\* For method of selection see

The following method of determining yield has been employed for several years at the University of Wisconsin. The determinations are made in calibrated cylindrical cans. Each of these cans is equipped with

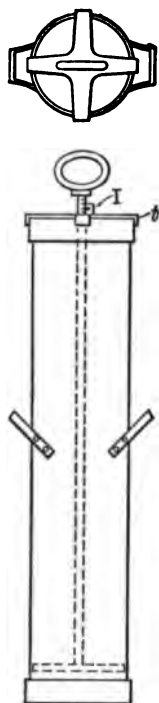


Fig. 3.—Yield Can.

a loosely fitting plunger fastened on the end of a piston rod which is so graduated that the volume of the mix may be read in decimals of a cubic foot. Fig. 3 shows the large yield can used for determining yield of concrete. About 3 lb. of dry materials are used to determine the yield of mortars and 150 lb. for the determinations on concrete. After the specific weight of the aggregate has been gotten, the requisite amount of each material is weighed and mixed on a non-absorbent mixing board in accordance with a standard method. The mix is then introduced into the cylinder in four layers of equal thickness. In the mortar tests each layer is compacted by an iron tamper having a face  $1\frac{1}{8}$  in. in diameter, which is the radius of the inside of the mortar yield can. In the tests on concrete, after the introduction of each layer, the material is puddled with an iron rod and the cylinder is raised 6 in. and dropped on the concrete floor five times to compact the mixture. After the mixing board and tools have been thoroughly cleaned and all material put into the can the plunger is inserted, and whirled around to level the top of the mix. A tripod (*t*) which carries an index (*I*) is next slipped over the end of the piston rod and rested on the top of the yield can. The reading on the plunger rod opposite the index on the tripod gives the volume of the mix.

In calculating the results of the test the losses are apportioned in accordance with the amount of each material passing a No. 74 sieve, since experiments have shown that practically all material lost can be washed through this sieve.

**464. Density \*** is the ratio of the absolute volume of the solid particles of cement plus aggregate to the volume of the resulting mortar or concrete. If *c*, *s*, and *g*, represent the absolute volumes of cement, sand, and gravel, respectively, in a unit volume of concrete, the density  $= \rho = c + s + g$ . If the specific gravities of the cement and aggregate are known then the density of a mix may be computed from the results of the yield test as follows: Let *V* = volume of mix; *W<sub>c</sub>*, *W<sub>s</sub>*, and *W<sub>g</sub>* = the weights of the cement, sand and gravel, respectively, corrected for losses in mix

\* Note that this definition differs from that usually given in treatises by excluding the mass of the water. The term solidity ratio, adopted in *Paper No. 58* of the U. S. Bureau of Standards, is more exact.



$K_c$ ,  $K_s$ , and  $K_g$  = the specific gravities of the cement, sand, and gravel, respectively;  $W_w$  = the weight per cubic unit of water, then

$$\rho = \frac{1}{W_w V} \left[ \frac{W_c}{K_c} + \frac{W_s}{K_s} + \frac{W_g}{K_g} \right]$$

Experiments have shown that the density of a mortar or concrete is a measure of its strength and imperviousness.

#### CHARACTERISTICS AND PROPERTIES OF FINE AGGREGATE

**465. Importance of Good Aggregate.**—Although the importance of testing cement has been recognized for a good many years, it is only recently that the necessity for ascertaining the properties of the aggregate has been appreciated. Now, however, with the results of numerous experiments and the records of failures caused by the use of poor aggregate before them, engineers are becoming convinced that a knowledge of the properties of the aggregate is fully as important as a knowledge of the properties of the cement. Inasmuch as fine aggregates of any one type, even those from the same deposit, often differ widely in their characteristics and properties, it is especially desirable to learn as much as possible about them before using in any important work. It must be remembered, however, that satisfactory test reports on the aggregate do not insure good concrete.

**466. Sampling Aggregate.**—Just as it is essential to secure representative material for cement tests and like analyses, so in determining the value of an aggregate, it is necessary that due attention be given to the securing of a fair sample. If the character of the deposit, or supply of material, is quite variable then individual samples of the different grades may well be taken and tested separately, or mixed together depending upon the degree of variation in quality.

In sampling the open face of a sand or gravel pit, the character of the overlying material, the stripping, should be noted and the face of the opening sampled from the bottom to the top. This may be conveniently done by scooping vertical troughs out of the face of the bank with a pail which should be so manipulated that an equal amount of material is secured from every portion of each trough. If sharply defined strata of fine and coarse particles are present, it is well to secure separate samples of the material composing these layers.

When one is sampling a pile of aggregate, it should be remembered that the coarse particles tend to separate and flow towards the bottom of the pile, consequently it is desirable either to flatten the pile and employ the "method of quartering"\* or, if the pile is very large, to proceed in the same manner as suggested in sampling a pit.

\* See Art. 401.

In selecting portions of a sample of aggregate for the different tests the method of quartering or an equally efficient method should be employed.

**467. Requirements for Fine Aggregate.**—The fine aggregate should consist of a mixture of hard, tough grains of different sizes. Particles of approximately equal dimensions are preferable to elongated grains; since the former, with similar gradation of sizes, produce more compact mixtures containing less voids than the latter. The sharpness of the grains is of little importance. The fine aggregate should be free from any minerals of a weak, friable nature and from other impurities such as organic or vegetable matter, the presence of which in very small percentages may be very injurious. The presence of 10 per cent or more of clay or loam is, in general, objectionable. Smaller percentages of these impurities may impair the efficiency of fine sands.

Roughly speaking, the sizes of the particles in fine aggregate suitable for concrete construction should be graded as follows: the maximum size of grain should not be over  $\frac{1}{4}$  in., between 40 and 60 per cent (by weight) should be held on a No. 30 sieve and not over 10 per cent should pass a No. 100 sieve.

Whenever time permits strength tests should be made of mortars containing the fine aggregate. In fact, it is unsafe to use sands of unproven quality unless such strength tests are made. A 1 : 3 mortar of standard consistency, made from good sand or screenings should have a strength at least equal to that of a similar mortar of standard sand made by the same operator. Compression tests are the most valuable, although tension or transverse tests are permissible. If the sand is to be used in making concrete it should be mixed with the proper proportions of the cement and stone with which it is to be used and the strength of the hardened mixture should be ascertained.\*

**468. Composition of the Particles.**—Although the results of a chemical analysis combined with a knowledge of the mineral content of a fine aggregate is sometimes helpful in determining its value, too great importance is often attached to the chemical analysis of sands. Some experimenters have believed that a very high silica content should be demanded in specifications for fine aggregate. Doubtless a sand which consists principally of hard, tough and non-porous grains of quartz is superior to a soft, calcareous sand, provided other characteristics are similar. Nevertheless there are many good sands and a variety of suitable broken stone screenings having comparatively low silica contents which make mortars of high strength.

Inasmuch as the strength of the mortar or concrete is dependent upon the strength of the aggregate as well as the strength of the cement the grains of sand or screenings should, at least, equal the cement in strength.

\* See Art. 485.

Since the resistance of cement paste to attrition and abrasion is small, the resistance of mortar to such influences is largely dependent upon the hardness and toughness of the aggregate. Consequently whenever high resistance to abrasion is desired the particles of the aggregate should be so hard that they cannot be readily scratched with a pocketknife. If the aggregate is very porous it is likely to withdraw water from the cement during the hardening. This results in the formation of a weak layer of paste surrounding the grains.

From a consideration of the above facts, it is, therefore, evident that the presence of a large proportion of hard, tough, dense minerals which are not readily attacked by atmospheric conditions is a favorable indication of the quality of an aggregate. If porous, the aggregate should be thoroughly soaked with water before mixing with cement.

Some of the more desirable mineral constituents are quartz, feldspar, and hornblende. Among the objectionable minerals mica, talc and iron pyrites should be mentioned. Since finely pulverized silica and limestone when mixed with water have cementitious properties, small percentages of these powders are not objectionable.

**469. Impurities.** The effects of natural impurities like clay and loam are dependent upon 1, the gradation of the sand particles; 2, the richness of the mortar; 3, the chemical constitution and the fineness of the particles in the clay or loam. The addition of clay or loam to a coarse, natural sand often so improves the gradation of the particles that a decrease in voids is produced. Therefore it may be possible to increase the density and strength of a lean mortar made of a coarse sand by adding small percentages of these impurities. More fine material of this character can be allowed in lean mixtures than in rich mixes, since in the latter a larger proportion of the fine particles needed for a mixture of maximum density is provided by the cement itself.

Besides filling voids, finely pulverized clay when mixed with water forms colloids, thereby furnishing additional strength to the resulting mortar. A rough qualitative test for detecting small percentages of loam or clay may be made by rubbing the sand between the palms and noting the amount and character of the stain thus produced. If present in large percentages clay imparts a greasy, slippery feeling to the sand when it is wetted and loam is generally readily detected by its dark color.

Generally, organic and vegetable impurities, the presence of which often greatly injure or destroy the setting power of the cement, form a part of the silt.

Therefore, sands containing 2 per cent of silt should be viewed with suspicion, and mortar strength tests should be made before the sand is used on the work.

In 1916 Messrs. D. A. Abrams and O. E. Harder perfected a colori-

metric tests for organic impurities in sands which may be carried out in the field as follows:

"Fill a 12-oz. graduated prescription bottle to the 4½-oz. mark with the sand to be tested. Add a 3 per cent solution of sodium hydroxide until the volume of the sand and solution, after shaking, amounts to 7 oz. Shake thoroughly and let stand over night. Observe the color of the clear supernatant liquid. . . . If the clear supernatant liquid is colorless, or has a light yellow color, the sand may be considered satisfactory in so far as organic impurities are concerned. On the other hand, if a dark-colored solution, ranging from dark reds to black is obtained the sand should be rejected or used only after it has been subjected to the usual mortar strength tests.

"Field tests made in this way are not expected to give quantitative results, but will be found useful in: (1) Prospecting for sand supplies; (2) Checking the quality of sand received on the job; (3) Preliminary examination of sands in the laboratory." \*

**470. Gradation of the Sizes of the Particles.**—Experiments have indicated that mortars made from fine sands are less dense and strong than those made from coarse sands. This is probably due to two causes: first, the difficulty of coating very fine particles of sand with cement particles of like size and second, the larger percentage of voids in the mixture of fine sand and cement. Experiments have also shown that a proper gradation of the sizes of the particles produces maximum strength, density and imperviousness. The most effective gradation of the sizes of the particles is dependent upon the richness of the mix and the maximum size of the aggregate.

Since a valuable index of the gradation of the sizes of the particles is afforded by mechanical analyses diagrams some representative curves will next be considered. Fig. 4 shows typical mechanical analyses curves for different grades of sand and screenings. Further information concerning these aggregates may be found in Table 1, Ch. XIV, Sand Sd. 1, which may be classed as a medium sand, is well-graded for making concrete but contains too many particles of fine and medium size to produce a lean mortar of maximum strength. Sand Sd. 13 is properly graded to make a 1 : 3 mortar of great strength but is too coarse to secure maximum strength when used in making concrete. Sand Sd. 21 is too fine for making either concrete or mortar. Sands graded like No. 21 too often constitute the major portion of the supply in a locality. Beach sands are generally of this class and, therefore, cannot be effectively used for concrete construction. Sands graded similarly to Sd. 1 are often found in pits, banks, and in river bottoms. Coarse sands like Sd. 13 are of rare occurrence.

Broken stone screenings should be free from large percentages of fine crusher dust especially if the latter is mixed with clay. In general, not

\* For laboratory methods of quantitatively determining the organic material present in sands see Circular No. 1, Structural Materials Research Laboratory of Lewis Institute, from which the above description was taken.

over 25 per cent should pass a No. 50 sieve and less than 10 per cent should go through the No. 100 sieve. The presence of a large proportion of fine dust is objectionable for the reason advanced against fine sands. Also if screenings are to be used for surfacing floors, sidewalks, or roadways, which are subjected to attrition the presence of an excessive proportion of

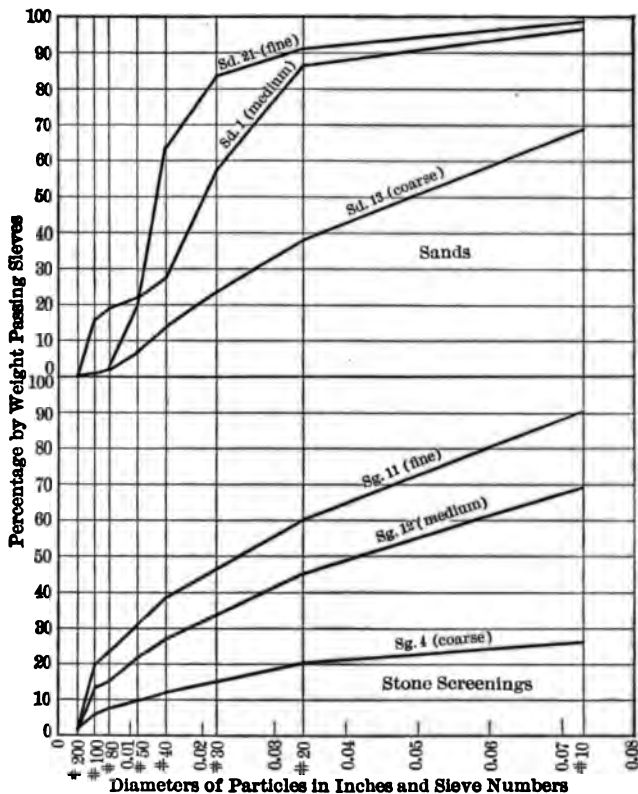


FIG. 4.—Mechanical Analyses Curves for Typical Sands and Screenings. (Bull. No. 331, U. S. Geol. Survey.)

fine powder is particularly objectionable; since mortars containing such aggregate "dust" and wear badly.

**471. Voids and Specific Weight.**—Two other criteria of the worth of sands and screenings are afforded by the percentage of voids and the specific weight. Approximately, we may say that for fine aggregates of like chemical and mineral constitution those with the lowest percentage of voids or the highest specific weight are the most desirable, since a less proportion of cement will be required to fill the unoccupied space in such aggregate. One must bear in mind, however, that due to differences in

gradation two sands which contain the same percentage of voids may produce mortars differing considerably in density and strength. For instance, comparing a fine and a coarse sand containing the same proportion of voids, the coarse sand will produce the stronger mortar.

Since even a small percentage of moisture tends to hold apart the grains and increase the voids, the moisture content exercises a very important influence upon the percentage of voids and the specific weight. For examples examine Fig. 5, which shows the effect of the percentage of moisture

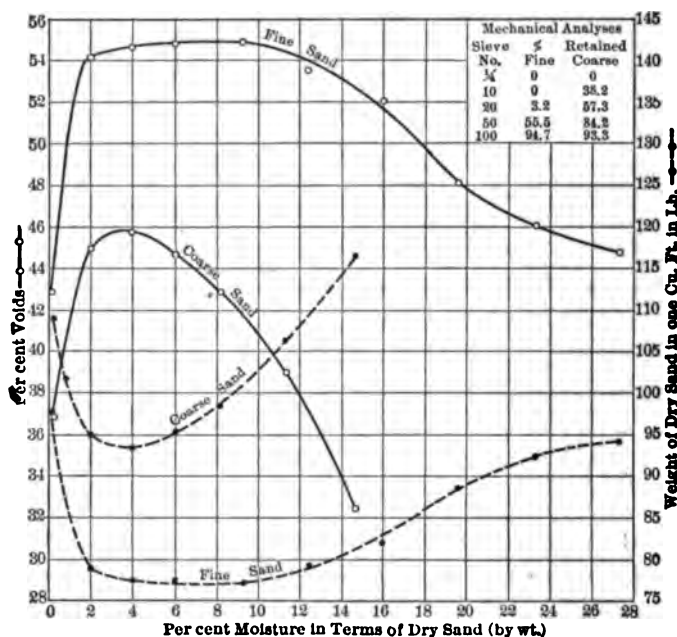


FIG. 5.—The Effects of Moisture on the Voids and Specific Weight of a Fine and a Coarse Sand.

upon the voids and specific weights for a fine and a coarse sand. The curves in the diagram serve to illustrate the importance of making the determinations on material containing the per cent moisture ordinarily present in field operations, if the results are to be employed in the field. If, however, a comparison of different sands is to be made determinations should also be conducted upon dried aggregate.

To facilitate the computation of the percentage of voids from the specific weights, the diagram in Fig. 6 is provided. Since a knowledge of the specific gravity of the aggregate is necessary in order to use this diagram the following average values are given for sands: Quartz, 2.65; dolomitic sands, 2.65–2.75; calcareous sands, 2.60–2.70. A rough average

value for all sands is 2.65. The specific gravities of stones may be found in Art. 268.

For good sands the percentage of voids generally lies between 28 and 35. The corresponding weights per cubic foot range from 120 to 105 lb. In screenings, on account of the angular shape of the particles, a somewhat

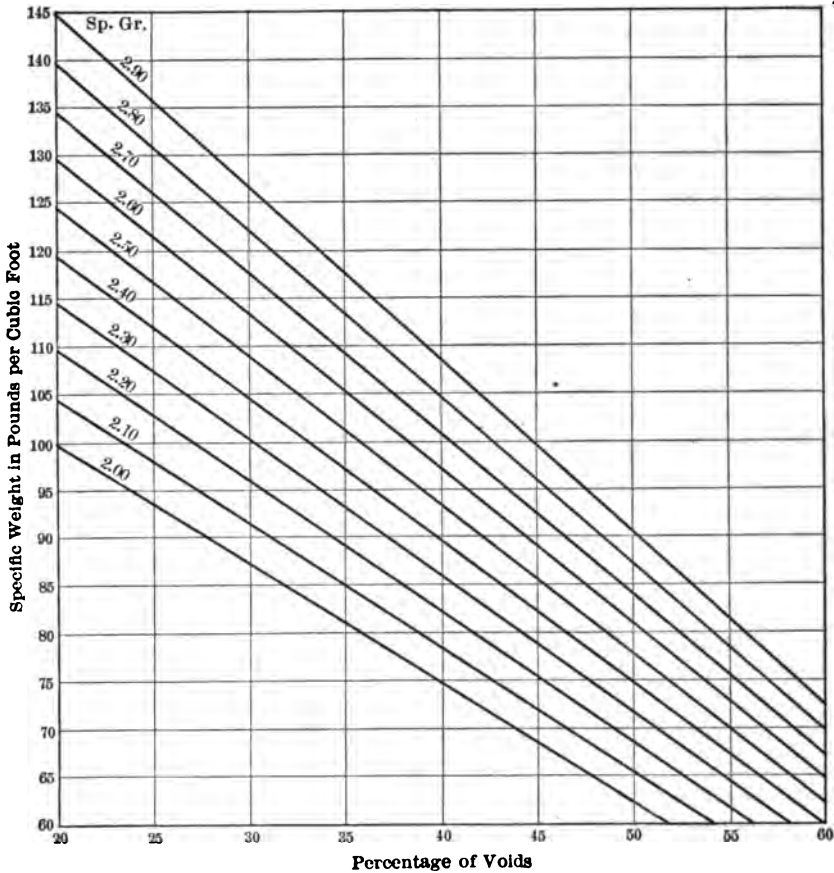


FIG. 6.—Relation of Specific Weight to Voids.

higher range in the percentage of voids is to be expected. Screenings of good quality should not, in general, contain over 38 per cent voids.

**472. Mortar Tests.**—By far the most valuable indices of the efficiency of a fine aggregate are given by the strength and yield tests of mortars made with a cement of known properties. Although tensile tests on 1 : 3 mortar briquettes are ordinarily made, compression tests are of more value. Cross-bending tests are of some value but should only be employed where facilities for testing are limited. The yield test is often made to ascer-

tain both the yield and density of the mortar. The density forms a much more valuable indication of the probable strength of the mortar than can be obtained from a determination of the voids in the fine aggregate.

In making these tests a standard consistency should be adopted for the mortars under comparison. Some experimenters believe in employing the standard consistency used in cement testing, while others are in favor of a slush consistency comparable to that used in construction. The advocates of the latter claim that the proportionate strength of poor sands is much less for the slush consistency than for the normal. Consequently poor material is more readily detected when the wet consistency is employed.\* On the other hand, if density is to be determined, the use of a slush consistency in yield tests produces misleading results due to the excess of water.

A rough but satisfactory method of determining normal consistency for mortars made of fine aggregate and cements follows. Weigh the quantities of standard sand and cement necessary to make 500 or 600 grams of dry material of the required proportions, mix dry and form into a crater in accordance with the method suggested in Art. 413. Do likewise with a similar quantity of the given sand or screenings and cement. To the standard sand mixture add the proper amount of water to bring the mortar to normal consistency and mix in the standard way prescribed in Art. 413. Next estimate the necessary percentage of water to bring the test mortar to the same consistency, and mix. As an aid to such estimation it should be remembered that a natural sand or screenings will generally require from 10 to 50 per cent more water than standard sand and that the percentage of water required varies with the proportion of fine material in the aggregate. Two or three trials will ordinarily suffice to obtain the consistency which appears and feels like that of the standard sand mortar.

Mortars of normal consistency suitable for construction purposes should possess strengths at least equal to standard sand mortars. In general, the density of a 1 : 2 or 1 : 3 mortar of standard consistency made of well-graded sand will lie between 0.70 and 0.75; the yield will generally vary between 1.08 and 1.20 for similar mixes, being greater for the richer mix.

#### CHARACTERISTICS AND PROPERTIES OF COARSE AGGREGATE

**473. Requirements for Coarse Aggregate.**—Any stone or gravel possesses the strength of neat cement is sufficiently strong for aggregate. In hardness, however, there is considerable difference depending upon the kind of construction. Other surfaces subjected to considerable

\* See *Proc. A.S.T.M.*, Vol.



dressing is placed a uniformly hard and tough coarse aggregate which cannot be scratched by the knife is desired. For other forms of construction extreme hardness is not essential provided the aggregate is tough and strong.

The coarse aggregate should also be free from loam, clay, vegetable or organic matter, and other injurious substances previously mentioned in discussing fine aggregates.

Preferably, the particles of the aggregate should be approximately cubical or spherical in form. Flat, disc-shaped pieces and long, thin wedge-like particles are objectionable since they cannot be so closely compacted as the cubical or rounded stones. To secure a good bond between the mortar and the coarse aggregate, the cleavage planes of broken stone and the surfaces of gravel particles should be uneven and rough. Very porous particles must be saturated with water before mixing with cement.

The size of the particles of the coarse aggregate should either be uniform or else uniformly graded from fine to coarse. Material passing a  $\frac{1}{4}$ -in. sieve should either be screened out or, if it is of proper quality, it may be substituted for an equal amount of fine aggregate. Pit-run gravel or other mixed aggregate in which the maximum size of particle is 1 in. should not contain over 50 per cent, by weight, of material passing a  $\frac{1}{4}$ -in. sieve. The maximum permissible size of stone or gravel particles depends upon the character of the construction. For massive work such as heavy piers, abutments, thick walls, footings and similar constructions, aggregate passing a  $2\frac{1}{2}$ -in. mesh is often used; but for thin walls, reinforced beams and columns, and small members in general, material passing a 1-in. sieve is frequently specified.

The voids in well-shaped broken stone of uniform size generally vary between 45 and 55 per cent. In uniformly graded broken stone, the per cent voids should lie between 40 and 50 depending upon the range in sizes. The per cent voids in well-graded gravel above  $\frac{1}{4}$  in. in size should not be over 38. In well-graded run-of-pit material the voids should not be above 28 per cent.

**474. Characteristics and Properties of Broken Stone.**—The broken stones commonly used for coarse aggregate are trap rock, granite, dolomite, limestone and sandstone. Trap rock, on account of its hardness, toughness, and great strength, is an excellent coarse aggregate. Furthermore, the real trap rocks possess a greater resistance to high temperatures than the other broken stones. The granites also furnish a very good material for coarse aggregate. Both of these classes of rock are very desirable in concrete or floor construction, or on surfaces which are subjected to considerable abrasion. Of them, however, the hardest igneous rocks crush into concrete or mortar into which

they are made. When such aggregate have very smooth cleavage planes, the mortar will not strongly adhere to the surfaces and the resulting concrete will be deficient in strength. In spite of these objections, however, these rocks form the most valuable class of crushed aggregates.

Crushed dolomite and limestone are extensively used for coarse aggregate, and the hard varieties of these rocks form a good material for constructions requiring high strength. As a class, however, these aggregates have not as high resistance to fire or to abrasion as the traps and granites. Soft limestone is often very porous, and unless thoroughly wetted before mixing will weaken the strength of the surrounding mortar by absorbing water from it while the hardening process is going on. Limestones frequently contain large quantities of very fine dust which, in wet weather, coat the surfaces of the stones. If there is a large proportion of this dust in the coarse aggregate, and if a fine sand is being used, it may be necessary to screen out the dust in order to secure a concrete of high strength. However, if the sand is coarse, allowance may be made for this dust in proportioning.

The stronger and more dense sandstones make a satisfactory coarse aggregate, but the soft varieties together with the shales and slates should not be used. The latter are likely to be deficient in strength and to consist of particles which are improperly shaped for making dense concrete.

The voids in crusher-run of broken stone with the dust out generally vary between 43 and 50 per cent. The per cent voids varies with the degree of compacting, but is not materially affected by the way in which the stone is poured into the measure. Dropping from a considerable height tends towards greater compactness. Table 2 contains a summary of results obtained by Prof. I. O. Baker of the University of Illinois \* on the voids in different grades of crushed limestone. Table 3 has been compiled from various reports. The voids or weights per cubic foot for crushed stone may be gotten from Fig. 7; specific gravities of various classes of stone will be found in Art. 268.

The best gradation of sizes of coarse aggregate is in dispute. Some authorities claim that coarse broken stone consisting of particles of approximately the same size is preferable to aggregate which is graded from  $\frac{1}{4}$  in. to the same maximum size. On account of the tendency of coarse stone to separate from the concrete in which the coarse aggregate is of one size only, it seems preferable to use a graded mixture. Furthermore, experiments have shown that concrete made of coarse aggregate which varies uniformly in gradation is just as strong and dense as that made from one-size coarse aggregate. Fig. 7 shows typical mechanical analysis curves for crusher-run broken stone and the desirable form of curve for such aggregates. Since the coarser particles of stone roll to the

\* *Bull.* No. 23 Univ. of Ill. Eng. Expt. Sta., 1908.

to high temperatures have been employed considerably for making fire-proof concrete. For this purpose the cinders should be a hard, vitreous clinker, practically free from sulphur, iron oxide, and combustible substances. From the standpoint of fireproofing, a highly porous clinker is desirable. However, care should always be taken to thoroughly drench the clinker before mixing so that it will not absorb water from the mortar during the process of hardening. Cinders make a very light concrete but should not be employed in constructions demanding high strength or imperviousness to water.

Slag has been used to some extent both as fine and as coarse aggregate. It is an efficient aggregate for fireproofing. Often, however, the concrete made from it is deficient in strength. The slag sand is formed by running the molten stream from the furnace through a water spray into a large tank. The coarse material is sometimes obtained by crushing the refuse of the slag dump. Freedom from sulphur is also essential in slag aggregates and, like cinders, porous slag should be thoroughly wetted before mixing.

Tailings from zinc and lead mines have been considerably used in Missouri and southwestern Wisconsin for concrete aggregate. The chief objection to these aggregates is the presence of pyrites, which causes rust-like spots on the surface of the concrete.

#### THE PROPORTIONING OF MORTARS AND CONCRETES

**478. The Principles of Proportioning.**—The three most important laws of proportioning may be stated as follows:

1. The strength and impermeability of mortars and concretes of like density and the same constituent materials increase with the proportion of cement in the mixture provided the consistency is constant.

2. With the same proportion of cement, with like consistency and like materials the strength and impermeability increase with the density of the mixture.

3. Under common methods of placement the strength and impermeability of well-cured concrete and mortar are greatest for plastic or mushy consistencies. Mixtures of dry consistency, although strong at an early age, are generally porous. An excess of water produces a weak concrete which hardens more slowly than a plastic mix.

In addition to the above, the following statements appear to be justified by the results of experience and tests:

- (a) To proportion concrete for the maximum resistance to fire, a porous non-combustible aggregate of high specific heat together with cement sufficient to produce the requisite strength should be thoroughly mixed with a large percentage of water and placed with as little ramming as possible. Such concrete when hardened will be porous and contain a

high percentage of combined water. These properties are of great value in resisting the transfer of heat through fireproofing.

(b) In proportioning concrete or mortar which is to be subjected to freezing temperatures, a minimum amount of water should be used and a quick-setting cement.\* Lean mixtures appear to resist freezing better than rich ones.

(c) Since the resistance of concrete to abrasion is dependent upon the uniformity, hardness and toughness of the aggregate and the rigidity with which it is held in place, concrete for road construction should be made from a carefully graded, hard, tough aggregate bound together with as small a proportion of rich mortar as is consistent with the required strength and imperviousness.

Inasmuch as the density of a mixture is so often a criterion of other physical properties, the more scientific methods of proportioning are essentially schemes for securing maximum density. Two principal aids in proportioning are afforded by a knowledge of the percentage of voids and the mechanical analysis.

**479. The Measurement of Proportions.**—The most accurate method of measuring proportions is to weigh the required quantities of each material. This may be done whether the proportions are based upon volumes or weights. In Europe proportions are generally stated in the terms of weight, but in this country proportions are nearly always expressed as parts by volume. The objections to proportioning by weight are the indefiniteness due to variation in specific gravity of the aggregate and the delay caused by using scales. On the other hand, measuring by volumes, as now practiced, is not satisfactory owing both to the inaccuracies in the methods of measuring and to the large variations in the volume of fine aggregate resulting from small changes in the moisture content. Nevertheless in America despite the manifest inaccuracy, volumes of aggregate are generally measured by the wheelbarrowful. Whenever measurements are made by the wheelbarrow or by a similar method care should be taken to specify the size of barrow, how it shall be filled, and the allowance which shall be made for moisture in the sand. In short, the methods of measuring, as well as the unit of measurement should be carefully specified. Cement is usually measured by the bag, one bag containing approximately 1 cu. ft. and weighing 94 lb. In laboratory practice the weight of a cubic foot of cement is commonly assumed to be 100 lb.

**480. Arbitrarily Selected Proportions.**—Probably the most used method of proportioning is that based upon arbitrary selection. In specifying proportions for mortars it is common practice to call for 1 : 2, 1 : 3 or 1 : 4 parts of cement to parts of fine aggregate, depending upon the quality of mortar demanded. In proportioning concrete by this method,

\* See also Art. 359 and 539.

the engineer, guided by experience and a knowledge of the requirements for the structure, assumes a mortar of given proportions and selects a proportion of stone such that the voids will be filled with mortar. For concrete made of coarse aggregate containing large percentages of voids, proportions of cement, sand and coarse aggregate similar to the following are often called for: 1 : 2 : 3, 1 : 3 : 5. If the coarse aggregate contains a normal percentage of voids, say 45 to 50—1 : 1 : 2, 1 : 2 : 4, 1 : 3 : 6, 1 : 4 : 8 and like proportions are frequently specified. With an aggregate containing still less voids such as a well-graded crushed stone or gravel 1 : 1½ : 3½, 1 : 2 : 5, 1 : 3 : 7 are sometimes selected. Obviously such methods or proportioning are crude and do not lead to the most efficient use of materials. They may give very fair results when used by a man experienced in judging aggregate, but in the hands of a person inexperienced in their use a poor mix or a waste of cement is likely to result.

**481. Proportions Based on Voids.**—A great variety of rules for proportioning based upon the principle of filling the voids in the aggregate with cement have been formulated. There are two main reasons why this method is not accurate. First, the problem of wetting the aggregate and bringing it into the same state of compactness which it assumes in the concrete or mortar is very difficult of solution; second, the general assumption that the particles of cement will fit into the void spaces in aggregate is fallacious, especially if particles passing a No. 50 sieve are present in the aggregate.

In proportioning mortars, the common application of the above method is to determine the voids in the sand or screenings in the condition in which they are to be used and then find the amount of neat cement paste required to fill the voids. For the latter determination about 112 lb. of cement should be allowed for 1 cu. ft. of paste. In general, this method does not lead to a determination of the densest mixture nor the most effective proportions of cement and sand. It is especially unsatisfactory with very fine sand or screenings.

One rule advocated for proportioning concrete is to fill the voids in the coarse aggregate with sand and the voids in the latter with cement. Another rule suggests filling the voids in the coarse aggregate with a slight excess of mortar. A third rule recommends filling the voids in the mixed aggregate with cement. None of these methods is applicable to all classes and gradations of materials with equal efficiency. They are all faulty if there are particles of the same size in either the cement and sand or in the sand and coarse aggregate. Experiments have been made indicating that such rules do not enable one to secure concrete of the maximum strength and density with a minimum percentage of cement. The error lies in the *assumption* that under the above conditions the finer particles of material will occupy the interstices in the larger. Whereas the

aim should be to select such proportions of cement, sand and stone that the resultant mixture will be the densest. In general we may say that the above methods of proportioning by void filling yield results which are very little better than those obtained by arbitrary selection.

**482. Proportions Based on Minimum Yield.**—A cut-and-try method of proportioning which possesses considerable merit is that based upon a minimum yield with a given proportion of cement to aggregate. Since the proportion of cement must be based upon judgment, the chief use of this method is to secure the most efficient combination of the different grades of aggregate. Trial mixtures of cement and different grades of aggregate are mixed together on a non-absorbent platform and sufficient water added to form a plastic consistency. The yields are then determined in the ordinary way (see Art. 463). Evidently, if the weights of cement, aggregate and water are kept constant and the specific gravities of the different grades of aggregate are the same, the mixture having the least volume will be the densest.

Let us briefly consider the application of this method to proportioning a 1 : 3 mortar, proportions by weight. Suppose that the sand passes a  $\frac{1}{4}$ -in. sieve and has been divided into two grades by an  $\frac{1}{8}$ -in. sieve. We would next assume the following trial proportions:

TRIAL NO.	PROPORTIONS.		
	Cement.	Fine Sand.	Coarse Sand.
1	1	1	2
2	1	$1\frac{1}{2}$	$1\frac{1}{2}$
3	1	2	1

From the volumes of mortar gotten in these tests other desirable proportions may be estimated and tried. If, for instance, the second trial above gave the minimum volume and the third gave the next lowest, then it would be well to try a  $1 : 1\frac{3}{4} : 1\frac{1}{4}$  mix.

The efficiency of this method of proportioning is well shown by the results of Feret's Experiments given in Art. 502. The remarkable similarity of the triangular diagram, representing the variation in density, to the diagrams showing the variation in compressive strength of mortars made from different gradations of sand forms a very convincing proof of the value of this method for mortars.

Trial mixtures for proportioning concrete may be estimated as in the case of mortar. For example, if cement, sand, and gravel are to be used in a 1 : 7 mix, the mixes tabulated below will serve in the preliminary trials.

TRIAL NO.	PROPORTIONS.		
	Cement.	Sand.	Gravel.
1	1	1½	6
2	1	2	5
3	1	3	4

For graded aggregate the following variation of the above method has been successfully employed by the Warren Bros. Bituminous Paving Co.\* It advocates the use of a graded aggregate containing in compact volume less than 21 per cent voids. In securing a dense aggregate use is made of a yield can shaped like a conical frustum with the large end at the bottom. This can is filled with the coarsest aggregate and thoroughly compacted by raising it off the ground and letting it fall a number of times. When no more of the coarsest material can be placed in the can, the mixture is emptied, an addition of the next finer material is made, and the material is again jostled. This cycle of operations is repeated until no more of this second grade of aggregate can be put into the can and the third and remaining sizes are introduced in a similar manner. Mr. A. E. Schuette, who invented this method, found it satisfactory for all classes of aggregate varying in size from 3 in. down to the material passing a No. 200 sieve. For aggregates differing in shape of particles, he found the proportions of the different sizes of particles varied, but after having established the proper combination for a given set of materials he found it unnecessary to make any large alteration in proportions.

**483. Proportioning by Mechanical Analysis.**†—From the results of numerous experiments W. B. Fuller and S. E. Thompson ‡ advocate the grading of the aggregate into different sizes and combining them with the cement to form a mixture having a fixed mechanical analysis curve. Their tests indicate that concretes thus proportioned have maximum density, strength, and imperviousness. The curve which they propose is a combination of an ellipse and straight line tangent as shown in Fig. 8. To assist in drawing these curves, the table on page 431 was devised by the experimenters. In this table  $D$  is the maximum diameter of the stone,  $a$  is the length of the semi-major or horizontal axis of the ellipse which is parallel to the axis of sizes and distant therefrom 7 per cent,  $b$  is the length of the semi-minor or vertical axis of the ellipse and is measured upward from the point  $(a, 7)$ . The abscissa of the point of tangency is  $0.1D$ . The ellipse can be most easily drawn by the Trammel-point method.

\* H. P. Bowes in *Canadian Engr.*, Mar. 23, 1911. The method was devised by A. E. Schuette of that company.

† For a method of more general application see Appendix B.

‡ See *Trans. A.S.C.E.*, Vol. 59, p. 67, 1907.

## DATA FOR FULLER AND THOMPSON'S GRADATION CURVES.

Materials.	Intersection of Tangent on Ordinate at Zero Diameter, Per Cent.	Height of Tangent Point, Per Cent.	AXES OF ELLIPSE.	
			a In.	b Per Cent.
Crushed stone and sand. . . . .	28.5	35.7	0.150 <i>D</i>	30.4
Gravel and sand. . . . .	26.0	33.4	0.164 <i>D</i>	28.6
Crushed stone and screenings. .	29.0	36.1	0.147 <i>D</i>	30.8

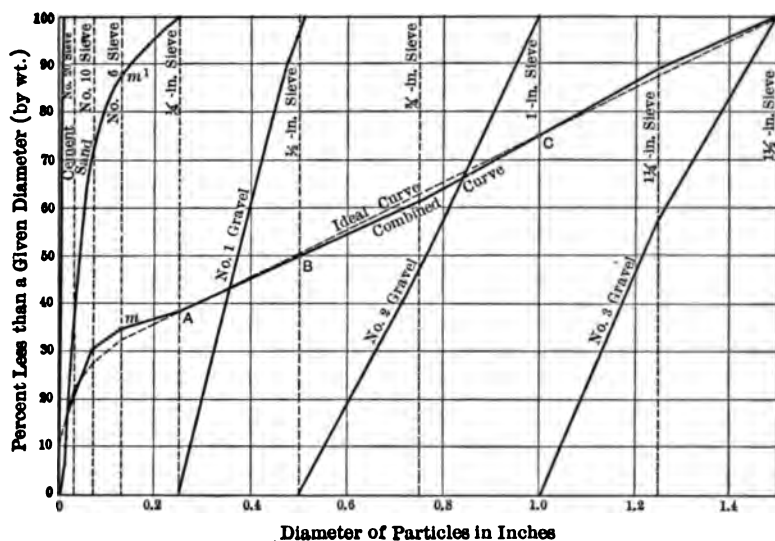


FIG. 8.—Fuller and Thompson's Method of Proportioning Concrete by Mechanical Analysis.

Cut-and-try methods of combining the various grades of material are employed. For instance, in Fig. 9 mechanical analyses curves are shown for the cement, sand and three grades of gravel. Let us suppose that it is desired to make a 1 : 9 concrete (1 part cement to 9 parts mixed aggregate by weight). An inspection of the curves for the different materials shows that the cement and sand must form the combined curve from 0 to  $\frac{1}{4}$  in., that gravel No. 1 must form the curve between  $\frac{1}{4}$  and  $\frac{1}{2}$  in., etc. The theoretical curve demands that the portion from point (0, 0), to A be filled with cement and sand. Therefore since 10 per cent of the entire weight is cement, the percentage of sand required will be the percentage corresponding to A minus 10, or 28. The percentage of No. 1 gravel required to fill the A B portion of the theoretical curve is 12; of No. 2 gravel for the B C portion is 25, etc. In drawing the combined curve the ordinate of



the point  $m = 10 + 28 \times 0.87$  (87 = the per cent corresponding to  $m'$ ) = 34.4 per cent. For overlapping curves the process is more laborious.

To successfully use this method of proportioning it is frequently necessary to separate the sand into at least two grades and the coarse aggregate into two or more grades depending upon the variation in sizes. These grades should not be remixed until they are placed in the mixer. From a study of the theoretical curve it is apparent that for rich mixes sand passing a No. 50 sieve should be eliminated, but in lean mixes some fine particles are beneficial. Experiments show that greater economy can be realized from the use of this method with lean mixes than with rich mixes. Fuller and Thompson in experiments on 1 : 9 mixes obtained an average gain in strength of about 14 per cent by grading the aggregate. Experiments at the University of Wisconsin \* on 1 : 9 mixes of gravel concrete have also demonstrated the beneficial effects of grading the aggregate on both the strength and imperviousness. Tests plainly show that a proper grading of the cement-sand portion in the combined mechanical analysis curve is of more importance than an exact grading of the coarse aggregate.

From the foregoing it appears that this method of proportioning will effect economy where the increased cost of screening, storing and handling the aggregate is less than the cost of the cement necessary to produce the same quality of concrete. In general, the method is only applied to large jobs.

**484. Proportions Commonly Used in Different Constructions.**—The proportions listed below are commonly specified when good materials are obtainable. Proportions marked with an \* refer to natural cement mortars.

**Mortar for:**

Laying brick and stone masonry.....	{ 1 : 2 to 1 : 3 1 : 1* to 1 : 2*
Filling joints in sewer pipe.....	{ 1 : 0 to 1 : 2 1 : 0* to 1 : 1*
Surfaces of floors, sidewalks and pavements.....	1 : 1 to 1 : 2
Waterproof linings.....	1 : 0 to 1 : 2
Cement bricks and blocks.....	1 : 2½ to 1 : 4

**Concrete for:**

Gravity retaining walls, heavy foundations and structures needing mass more than strength (compressive strength at 28 days = 1000 to 1500 lb./in. <sup>2</sup> ).....	1 : 3 : 6 to 1 : 4 : 8
Retaining walls, piers, sewers, pavement foundations, work requiring strength (compressive strength at 28 days = 1500 to 2000 lb./in. <sup>2</sup> ).....	1 : 2 : 4 to 1 : 3 : 6
Floors, beams, concrete pavements, reinforced concrete, arch bridges, low pressure tanks (compressive strength at 28 days = 2000 to 3000 lb./in. <sup>2</sup> ) <sup>1</sup> .....	1 : 1½ : 3 to 1 : 2½ : 4½

\* See *Jour. Western Soc. of Engr.*, Vol. 19, p. 813.

Reinforced concrete columns, conduit pipe, impervious concrete and work requiring great strength (compressive strength at 28 days = 3000 to 4000 lb./in.<sup>2</sup>) . . . . . 1 : 1 : 2 to 1 : 1½ : 3

**485. Testing the Quality of Concrete.**—Auxiliary tests on the aggregate are of assistance in making a selection of the proper aggregate, and the use of proper methods of proportioning tends toward economy in cement, but the value of these tests and schemes can only be ascertained by tests on the concrete itself. Such tests show that some fine aggregates, developing a low mortar strength, make very satisfactory concrete when mixed with coarse aggregate. Occasionally a coarse sand having a high mortar strength fails to produce as strong concrete aggregate as a finer grained sand when combined with a certain coarse aggregate. In making such tests, the sampling and proportioning of the materials, the method of mixing, the placing and the curing conditions for the specimens should all be controlled so that the test-pieces will in every way represent the concrete in the structure. For many structures, strength tests are of most value in determining the quality and uniformity of concrete. They should be made during the construction of every important concrete structure. Occasionally abrasion, absorption, fire or freezing tests are made. (For the methods used in making the latter tests see Ch. VIII and XIV.) We shall now briefly consider the strength testing of concrete.

A great variety of shapes and sizes of specimens has been employed in testing the compressive strength of concrete. In the United States practice favors either a 4- or 6-in. cube or a 6×12 or 8×16-in. cylinder. A cylinder is to be preferred to a cube for the reasons mentioned in Art. 113. When the capacity of the testing machine is not over 100,000 lb. the 6×12-in. cylinder is preferable. A larger cylinder should be used, however, when the maximum diameter of the coarse aggregate exceeds 2 in. Molds for the specimens should be non-absorbent and should fit tightly together. Means should be provided by which the ends of cylinders can be made plane and perpendicular to their axes. For this purpose it is well to set the molds upon heavy steel plates and to allow the specimen to partially set before leveling the top. Methods of bedding rough ends are suggested in Arts. 76 and 77.

Tests on the transverse strength can readily be made in simple machines of low capacity, consequently in the field where apparatus is limited the test can be conveniently used. It is to be hoped that some technical society will standardize this test and determine the proper specifications for different classes of concrete when it is used. A 6×6×42-in. beam loaded at the center over a 36-in. span will be found a satisfactory size of specimen for the transverse test of concrete. Since the modulus of rupture of concrete is greatly affected by shrinkage stresses, the test is only adapted to conditions where curing conditions are uniform. In testing,

care must be taken to make the loading arrangements in accordance with the principles of Art. 116.

On account of the curvature of the stress-deformation curve for concrete and owing to the shifting of the neutral axis toward the concave surface of the beam, the modulus of rupture computed from the flexure formula ( $S = \frac{Mc}{I}$ ), averages about 1.8 to 2.0 times the tensile strength.

Unfortunately, the ratio of the compressive strength to the modulus of rupture appears to be more variable, ranging from 4 : 1 to 8 : 1. The transverse test of plain concrete is not, therefore, an accurate method of determining the compressive strength of this material.

For either strength test there should be made, at least, three specimens for each variable covered. Results from seven-day tests are of value in ascertaining the strength developed by the mortar, but rarely is the strength of the latter great enough to shear the coarse aggregate. Tests at twenty-eight days and longer periods are more desirable.

**486. Quantities of Materials Required for One Cubic Yard of Mortar and Concrete.**—The values given in Table 4 are computed from the results of yield tests on 15 different sands and screenings.\* All of these aggregates were in air-dry condition at the time of test. It will be appreciated that the values will not apply to all mixes with equal accuracy, since the kind of cement, per cent moisture in aggregate, gradation of sizes, and percentage of particles passing a No. 100 sieve will considerably influence the yield. The greatest variations in yield are produced by variations in the moisture content in the aggregate. If the per cent moisture is known allowance for such variations may be made by using the information in Fig. 6 in connection with the yields in Table 4.

A rough estimate of quantities of materials required for a given volume of concrete may be gotten by computing the quantity of loose coarse aggregate which will fill the molds and base the computation of the sand and cement upon it. Thus, if the volume of the mold is 10 cu. ft. and the mix is 1 : 2 : 4, the above approximation calls for 10 cu. ft. of stone, 5 cu. ft. of sand and  $2\frac{1}{2}$  cu. ft. of cement. When bank-run materials or mixed aggregates are used this method generally gives results within 5 per cent. If the fine and coarse aggregates are separated it should only be used for rough estimates on small jobs, since for rich mixes the quantities may be 10 or 15 per cent too large.

A simple approximate formula applicable to the materials and proportions commonly used in making concrete is W. B. Fuller's rule,†

$$p = \frac{10.5}{c + s + g}.$$

\* *Proc. A.S.T.M.*, Vol. 13, p. 834.

† Taylor and Thompson's *Concrete Plain and Reinforced*, p. 16.

TABLE 4.—YIELD IN MORTAR FOR DIFFERENT PROPORTIONS OF VARIOUS AGGREGATES BASED ON LOOSE MEASUREMENTS OF VOLUME \*

All aggregates passed a  $\frac{1}{4}$ -in. sieve and were air dry. Plastic consistency.

No.†	Per Cent Voids.	PER CENT (BY WT.) PASSING SIEVE.				YIELD FOR MIX.			
		10	30	50	100	1 : 2	1 : 3	1 : 4	1 : 5
S ‡.....	37.0	100.0	0	0	0	1.18	1.11	1.08	1.07
Sd1.....	36.5	86.4	39.0	9.1	2.8	1.18	1.08	1.03	1.01
Sd2.....	35.2	81.2	61.4	34.5	9.7	1.25	1.15	1.10	1.07
Sd3.....	38.2	91.9	39.7	13.7	1.2	1.22	1.11	1.06	1.03
Sd4.....	39.8	100.0	95.8	62.5	8.6	1.30	1.15	1.09	1.04
Sd5.....	45.3	100.0	99.8	67.6	5.7	1.22	1.09	1.03	0.99
Sd7.....	36.6	67.7	25.9	11.2	2.2	1.19	1.07	0.99	0.95
Sd8.....	36.4	82.0	55.3	16.1	1.9	1.21	1.09	1.05	1.01
Sd9.....	36.0	66.8	13.9	4.9	1.4	1.14	1.03	0.98	0.95
Sd10.....	27.9	69.7	34.0	17.4	6.5	1.26	1.14	1.05	0.96
Sd11.....	35.0	72.0	23.8	4.3	0.5	1.21	1.08	1.01	0.99
Sg1.....	42.0	62.8	35.5	25.1	13.0	1.13	1.01	0.93	0.88
Sg3.....	39.0	64.0	28.0	19.5	13.8	1.15	0.99	0.92	0.88
Sg4.....	40.0	69.0	32.6	23.8	15.9	1.15	1.01	0.93	0.88
Tg1.....	43.6	34.6	15.4	11.5	7.5	1.07	0.97	0.89	0.84

\* Weight per cubic foot of cement was taken at 100 lb.

† Sd = sand; sg = screenings; Tg = zinc mine tailings.

‡ Standard Ottawa sand.

Here  $p$  = the number of barrels of cement required per cu. yd. of concrete. $c$  = the number of parts of cement by volume; $s$  = the number of parts of sand by volume; $g$  = the number of parts of gravel or broken stone by volume.Having  $p$ , the portion of a cubic yard of sand required per cubic yardof concrete is  $\frac{s}{c} \times \frac{4p}{27}$ ; the amount of gravel is  $\frac{g}{c} \times \frac{4p}{27}$ . In this com-

putation a barrel of cement is assumed to contain 4 cu. ft. For example, if 200 cu. yd. of 1 : 2 : 5 concrete are required for a certain structure, we shall find the quantities of materials needed.

$$p = \frac{10.5}{1+2+5} = 1.31 \text{ bbl.}$$

The required amounts are  $200 \times 1.31 = 262$  bbl. of cement,  $2 \times \frac{4}{27} \times 1.31$  $\times 200 = 77.6$  cu. yd. of sand;  $5 \times \frac{4}{27} \times 1.31 \times 200 = 194.1$  cu. yd. of gravel or broken stone.

More exact computations of quantities may be made by the use of the data in Table 5 which has been compiled from tests by Edwin Thatcher.

TABLE 5.—PROPORTIONS OF MATERIALS IN CEMENT CONCRETE, MODERATELY RAMMED

(From actual experiments made by EDWIN THATCHER, M. Am. Soc. C. E.)

CONCRETE WITH STONE 1 INCH AND UNDER.							CONCRETE WITH STONE 2½ INCHES AND UNDER.						
Proportions of Mixture.				Required for 1 Cubic Yard.			Proportions of Mixture.				Required for 1 Cubic Yard.		
Cement.	Sand.	Stone.	Ratio: Stone—Mortar.	Cement Barrels.	Sand, Cubic Yards.	Stone, Cubic Yards.	Cement.	Sand.	Stone.	Ratio: Stone—Mortar.	Cement Barrels.	Sand, Cubic Yards.	Stone, Cubic Yards.
1	1	2 0	1.42	2.57	0.39	0.78	1	1	2 0	1.42	2.63	0.40	0.80
1	1	2 5	1.79	2.29	0.35	0.88	1	1	2 5	1.79	2.34	0.36	0.82
1	1	3 0	2.14	2.06	0.31	0.94	1	1	3 0	2.14	2.10	0.32	0.96
1	1	3 5	2.50	1.84	0.28	0.98	1	1	3 5	2.50	1.88	0.29	1.00
1	1.5	2.5	1.40	2.05	0.47	0.78	1	1.5	2.5	1.40	2.09	0.48	0.80
1	1.5	3 0	1.68	1.85	0.42	0.84	1	1.5	3 0	1.68	1.90	0.43	0.87
1	1.5	3.5	1.97	1.72	0.39	0.91	1	1.5	3.5	1.97	1.74	0.40	0.93
1	1.5	4 0	2.25	1.57	0.36	0.96	1	1.5	4 0	2.25	1.61	0.37	0.98
1	1.5	4.5	2.53	1.43	0.33	0.98	1	1.5	4.5	2.53	1.46	0.33	1.00
1	2 0	3 0	1.38	1.70	0.52	0.77	1	2 0	3 0	1.38	1.73	0.53	0.79
1	2 0	3.5	1.61	1.57	0.48	0.83	1	2 0	3.5	1.61	1.61	0.49	0.85
1	2 0	4 0	1.84	1.46	0.44	0.89	1	2 0	4 0	1.84	1.48	0.45	0.90
1	2 0	4.5	2.07	1.36	0.42	0.93	1	2 0	4.5	2.07	1.38	0.42	0.95
1	2 0	5 0	2.31	1.27	0.39	0.97	1	2 0	5 0	2.31	1.29	0.39	0.98
1	2.5	3.5	1.37	1.45	0.55	0.77	1	2.5	3.5	1.37	1.48	0.56	0.79
1	2.5	4 0	1.57	1.35	0.52	0.82	1	2.5	4 0	1.57	1.38	0.53	0.84
1	2.5	4.5	1.76	1.27	0.48	0.87	1	2.5	4.5	1.76	1.29	0.49	0.88
1	2.5	5 0	1.96	1.19	0.46	0.91	1	2.5	5 0	1.96	1.21	0.46	0.92
1	2.5	5.5	2.16	1.13	0.43	0.94	1	2.5	5.5	2.16	1.15	0.44	0.96
1	2.5	6 0	2.36	1.07	0.41	0.97	1	2.5	6 0	2.36	1.07	0.41	0.98
1	3 0	4 0	1.34	1.26	0.58	0.77	1	3 0	4 0	1.34	1.28	0.58	0.78
1	3 0	4.5	1.51	1.18	0.54	0.81	1	3 0	4.5	1.51	1.20	0.55	0.82
1	3 0	5 0	1.68	1.11	0.51	0.85	1	3 0	5 0	1.68	1.14	0.52	0.87
1	3 0	5.5	1.85	1.06	0.48	0.89	1	3 0	5.5	1.85	1.07	0.49	0.90
1	3 0	6 0	2.02	1.00	0.46	0.92	1	3 0	6 0	2.02	1.02	0.47	0.93
1	3 0	6.5	2.18	0.96	0.44	0.95	1	3 0	6.5	2.18	0.98	0.44	0.96
1	3 0	7 0	2.35	0.91	0.42	0.97	1	3 0	7 0	2.35	0.92	0.42	0.98
1	3.5	5 0	1.48	1.05	0.56	0.80	1	3.5	5 0	1.48	1.07	0.57	0.82
1	3.5	5.5	1.63	1.00	0.53	0.84	1	3.5	5.5	1.63	1.02	0.54	0.85
1	3.5	6 0	1.77	0.95	0.50	0.87	1	3.5	6 0	1.77	0.97	0.51	0.89
1	3.5	6.5	1.92	0.92	0.49	0.91	1	3.5	6.5	1.92	0.93	0.49	0.92
1	3.5	7 0	2.06	0.87	0.47	0.93	1	3.5	7 0	2.06	0.89	0.47	0.95
1	3.5	7.5	2.21	0.84	0.45	0.96	1	3.5	7.5	2.21	0.86	0.45	0.98
1	3.5	8 0	2.36	0.80	0.42	0.97	..	..	..	..	..	..	..
1	4 0	6 0	1.57	0.90	0.55	0.82	1	4 0	6 0	1.57	0.92	0.56	0.84
1	4 0	6.5	1.70	0.87	0.53	0.85	1	4 0	6.5	1.70	0.88	0.53	0.87
1	4 0	7 0	1.83	0.83	0.51	0.89	1	4 0	7 0	1.83	0.84	0.51	0.90
1	4 0	7.5	1.96	0.80	0.49	0.91	1	4 0	7.5	1.96	0.81	0.50	0.93
1	4 0	8 0	2.10	0.77	0.47	0.93	1	4 0	8 0	2.10	0.78	0.48	0.95
1	4 0	8.5	2.23	0.74	0.45	0.95	1	4 0	8.5	2.23	0.76	0.46	0.98
1	4 0	9 0	2.36	0.71	0.43	0.97	..	..	..	..	..	..	..
1	5 0	9 0	1.94	0.66	0.50	0.90	1	5 0	9 0	1.94	0.67	0.52	0.93
1	5 0	10 0	2.15	0.62	0.47	0.95	1	5 0	10 0	2.15	0.63	0.48	0.96
1	6 0	11 0	2.04	0.55	0.51	0.93	1	6 0	11 0	2.04	0.56	0.52	0.94
1	6 0	12 0	2.22	0.52	0.48	0.95	1	6 0	12 0	2.22	0.54	0.49	0.98
1	7 0	13 0	2.07	0.47	0.50	0.93	1	7 0	13 0	2.07	0.48	0.51	0.95
1	7 0	14 0	2.23	0.45	0.48	0.96	1	7 0	14 0	2.23	0.46	0.49	0.98

PROPORTIONS OF MATERIALS IN CEMENT CONCRETE, MODERATELY  
RAMMED.—(Continued).

CONCRETE WITH 2½-INCH STONE, SCREENED.							CONCRETE WITH GRAVEL ½ INCH AND UNDER.						
Proportions of Mixture.				Required for 1 Cubic Yard.			Proportions of Mixture.				Required for 1 Cubic Yard.		
Cement.	Sand.	Stone.	Ratio: Stone Mortar.	Cement Barrels.	Sand. Cubic Yards.	Stone. Cubic Yards.	Cement.	Sand.	Gravel.	Ratio: Gravel Mortar.	Cement Barrels.	Sand. Cubic Yards.	Gravel. Cubic Yards.
1	1	2 0	1.42	2.72	0.41	0.83	1	1	2.5	1.79	2.10	0.32	0.80
1	1	2 5	1.79	2.41	0.37	0.92	1	1	3 0	2.14	1.89	0.29	0.86
1	1	3 0	2.14	2.16	0.33	0.98	1	1	3 5	2.50	1.71	0.26	0.91
...	...	...	...	...	...	...	1	1	4 0	2.86	1.55	0.24	0.94
1	1.5	2.5	1.40	2.16	0.49	0.82	1	1.5	3.0	1.68	1.71	0.39	0.78
1	1.5	3.0	1.68	1.96	0.45	0.89	1	1.5	3.5	1.97	1.57	0.36	0.83
1	1.5	3.5	1.97	1.79	0.41	0.96	1	1.5	4.0	2.25	1.46	0.33	0.88
1	1.5	4.0	2.25	1.64	0.38	1.00	1	1.5	4.5	2.53	1.34	0.31	0.91
...	...	...	...	...	...	...	1	1.5	5.0	2.81	1.24	0.28	0.94
1	2 0	3 0	1.38	1.78	0.54	0.81	1	2 0	3 5	1.61	1.44	0.44	0.77
1	2 0	3 5	1.61	1.66	0.50	0.88	1	2 0	4 0	1.84	1.34	0.41	0.81
1	2 0	4 0	1.84	1.53	0.47	0.93	1	2 0	4 5	2.07	1.26	0.38	0.86
1	2 0	4 5	2.07	1.43	0.43	0.98	1	2 0	5 0	2.31	1.17	0.36	0.89
...	...	...	...	...	...	...	1	2 0	6 0	2.77	1.03	0.31	0.94
1	2 5	3 5	1.37	1.51	0.58	0.81	1	2 5	4 0	1.57	1.24	0.47	0.75
1	2 5	4 0	1.57	1.42	0.54	0.87	1	2 5	4 5	1.76	1.16	0.44	0.80
1	2 5	4 5	1.76	1.33	0.51	0.91	1	2 5	5 0	1.96	1.10	0.42	0.83
1	2 5	5 0	1.96	1.26	0.48	0.96	1	2 5	5 5	2.16	1.03	0.39	0.86
1	2 5	5 5	2.16	1.18	0.44	0.99	1	2 5	6 0	2.36	0.98	0.37	0.89
...	...	...	...	...	...	...	1	2 5	7 0	2.75	0.88	0.33	0.93
1	3 0	4 0	1.34	1.32	0.60	0.80	1	3 0	5 0	1.68	1.03	0.47	0.78
1	3 0	4 5	1.51	1.24	0.57	0.85	1	3 0	5 5	1.85	0.97	0.44	0.81
1	3 0	5 0	1.68	1.17	0.54	0.89	1	3 0	6 0	2.02	0.92	0.42	0.84
1	3 0	5 5	1.85	1.11	0.51	0.93	1	3 0	6 5	2.18	0.88	0.40	0.87
1	3 0	6 0	2.02	1.06	0.48	0.97	1	3 0	7 0	2.35	0.84	0.38	0.89
...	...	...	...	...	...	...	1	3 0	7 5	2.52	0.80	0.37	0.91
...	...	...	...	...	...	...	1	3 0	8 0	2.68	0.76	0.35	0.93
1	3 5	5 0	1.48	1.11	0.59	0.85	1	3 5	6 0	1.77	0.88	0.46	0.80
1	3 5	5 5	1.63	1.06	0.56	0.89	1	3 5	6 5	1.92	0.83	0.44	0.82
1	3 5	6 0	1.77	1.00	0.53	0.92	1	3 5	7 0	2.06	0.80	0.43	0.85
1	3 5	6 5	1.92	0.96	0.51	0.95	1	3 5	7 5	2.21	0.76	0.41	0.87
1	3 5	7 0	2.06	0.91	0.49	0.98	1	3 5	8 0	2.36	0.73	0.39	0.89
...	...	...	...	...	...	...	1	3 5	8 5	2.51	0.71	0.38	0.91
...	...	...	...	...	...	...	1	3 5	9 0	2.65	0.68	0.36	0.92
1	4 0	6 0	1.57	0.95	0.58	0.87	1	4 0	7 0	1.63	0.77	0.47	0.81
1	4 0	6 5	1.70	0.91	0.55	0.90	1	4 0	7 5	1.96	0.73	0.44	0.83
1	4 0	7 0	1.83	0.87	0.53	0.93	1	4 0	8 0	2.11	0.71	0.43	0.86
1	4 0	7 5	1.96	0.84	0.51	0.96	1	4 0	8 5	2.23	0.68	0.42	0.88
1	4 0	8 0	2.11	0.81	0.49	0.98	1	4 0	9 0	2.36	0.65	0.40	0.89
...	...	...	...	...	...	...	1	4 0	9 5	2.49	0.63	0.38	0.91
...	...	...	...	...	...	...	1	4 0	10 0	2.62	0.61	0.37	0.93
1	5 0	8 0	1.72	0.74	0.57	0.91	1	5 0	10 0	2.15	0.57	0.43	0.87
1	5 0	9 0	1.94	0.70	0.53	0.96	1	5 0	12 0	2.58	0.51	0.38	0.92
1	6 0	9 0	1.67	0.65	0.59	0.89	1	6 0	12 0	2.22	0.48	0.44	0.88
1	6 0	10 0	1.85	0.62	0.56	0.93	1	6 0	14 0	2.59	0.43	0.40	0.92
1	7 0	11 0	1.75	0.54	0.58	0.91	1	7 0	14 0	2.22	0.42	0.44	0.88
1	7 0	12 0	1.91	0.52	0.55	0.95	1	7 0	16 0	2.54	0.38	0.40	0.92

**487. Interpretation of the Meaning of Proportions.**—Sometimes it becomes necessary to substitute a pit-run gravel or other mixed aggregate when the specifications call for separate measurement of the fine and coarse aggregate. Since such substitution often causes disagreement, it should be provided for in the specifications. To illustrate, consider a 1 : 2 : 4 mix of cement, sand and gravel measured by volume. It has been erroneously argued many times that the equivalent proportions with pit-run gravel are 1 : 6, whereas volume measurements of 2 : 4 mixture of sand and gravel will generally show that about 1 : 5 or even a richer proportion is the proper equivalent. Furthermore, unless the pit-run material is well graded, it is likely that the quality of the substituted mix will be inferior to the specified even though the equivalent ratio of cement to aggregate is used.

### MIXING, PLACING, AND CURING

**488. Principles of Proper Mixing.**—The first consideration in mixing either mortar or concrete should be to bring all materials into a homogeneous mixture of uniform consistency in the minimum amount of time and without waste. Whatever method of mixing is used, in order to insure that in the completed batch each grain of aggregate will be coated with cement paste, the aggregate and cement should be dry mixed for a short time. The object of this dry mixing is to evenly distribute the cement throughout the mass and to prevent it from balling-up when the water is added. Since the time of mixing may be somewhat shortened if dry sand is employed, it may be economical on large jobs to cover the storage pile. On the other hand, a porous aggregate should not be allowed to completely dry since it will absorb water from the cement paste and adversely affect the hardening properties of the mix. Instances in which this action has caused a failure of the structure have been recorded.

The question of the proper consistency is largely dependent upon the character of the work. At the present time variations from a dry consistency which will barely show moisture under heavy ramming to a soupy mix which can be spouted into the molds are in use. The general practice in this country is to employ a wet mix which will readily flow for nearly all reinforced concrete construction. In European countries practice favors a somewhat drier consistency. The effects of different consistencies on the properties of mortar and concrete are discussed in Art. 503, 511 and 527.

**489. Hand Mixing.**—Mortar can be hand mixed most satisfactorily in a tight wooden or sheet-metal box. For a 3- or 4-wheelbarrow batch a box 4×8 ft. with sides 8 or 10 in. high is convenient. The sand is spread in a layer of uniform thickness over the bottom of the box and covered by a similar layer of cement, or if the batch is large the cement may be

sandwiched between equal layers of sand. The dry materials are then thoroughly mixed by hoe or by shovel until the mass is of uniform appearance.

If shovels are employed the men should work in pairs, partners facing each other on opposite sides of the box. Beginning at one end and working toward the other, both men shovel simultaneously into the pile giving each shovelful a flip which scatters the material as it falls. The entire pile should be given at least three turns in this manner. A long crater is next formed in the pile and filled with water. The batch should then be given at least three more turns with the shovels.

Mixing by hoe is not so effective as by shovels, if two or more men are available. If the hoe is employed, water is generally added at the end of the box, the dry mix is rapidly drawn down into it, and the whole mass vigorously worked until the consistency is uniform.

For hand mixing of concrete, a tight platform somewhat larger than the mortar box is desirable, and the mixing should be done with shovels. Two methods of procedure are effective. In either method the sand and wetted stone are spread in long, flat piles parallel to each other and about 3 ft. apart. The sand is covered with an even layer of cement, and the dry mortar given at least three turns. In one method the dry mortar is then spread over the wetted stone and two or three additional turns given the mass. A long crater is then formed in the top of the pile and the proper amount of water added. The edges of the pile are gradually turned into the crater until the water has been absorbed and the mixing is finished by three more turns. In the second method the dry mortar is tempered with enough water to make a soft consistency and turned twice. It is then spread over the wetted stone and the batch is given three more turns.

By the first method the final consistency of the batch can be more accurately gauged than by the latter. In the second method, however, the batch receives more mixing after the water has been added.

**490. Machine Mixing.**—Most concrete is now mixed by machine. Although there are a great many designs and forms of mixers, they may all be separated into two classes—batch mixers and continuous mixers. In the operation of a batch mixer a definite charge of materials is mixed and discharged before another batch is admitted. The other type of mixer receives and discharges material continually.

In general, concrete made in a batch mixer is more uniform than the product of the continuous mixer for the following reasons: All portions of the charge are mixed together in the batch machine, whereas in the continuous mixer product the uniformity of successive portions depends upon the regularity of the feeding device. Furthermore, in most types of continuous mixer the time of mixing is too short for thorough work, whereas in the batch machine the mixing period can be regulated as desired.



In selecting a mixer, especial attention should be given not only to the quality of the product and the initial cost of the machine but to other factors which vitally affect efficiency, cost of operation and maintenance, such as—1, time required in mixing; 2, waste in charging and discharging; 3, rapidity of charging and discharging; 4, ease in cleaning; 5, durability of mechanical parts; 6, capacity of the power drive; 7, visibility of charge during mixing; 8, accuracy of water-feeding device. In general the capacity of a batch mixer should be a little greater than can be handled by the gang employed.

In mixing with batch machines, it is desirable to admit the sand, cement and stone in the order named and mix dry for at least 10 or 15 sec. The water should then be rapidly added and the mixing continued for at

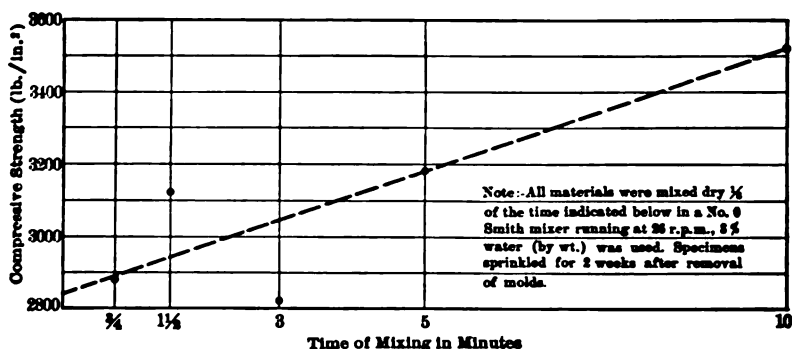


FIG. 9.—The Effect of the Length of Time of Mixing on the Compressive Strength of 1 : 2 : 4 Concrete. Age two months.

least one minute. When damp sand is being used and a very uniform concrete desired the above mixing period should be doubled. Experiments made at the University of Wisconsin \* show that the degree of imperviousness of 1 : 9 gravel concrete can be considerably increased by raising the time of mixing from  $\frac{3}{4}$  min. to 2 min., especially if damp sand is used. Fig. 9 shows the effect of increasing the time of mixing on the strength of 1 : 2 : 4 broken stone concrete. The limestone used in these tests passed a  $1\frac{1}{2}$ -in. screen and was retained on a  $\frac{1}{2}$ -in. sieve. The concrete was mixed in a No. 0 Smith mixer running at 28 to 30 r.p.m.

In Fig. 10 are plotted some results from tests by Prof. H. H. Scofield.† The test-pieces were made in a cube mixer running at 26 r.p.m. The age of the specimens was one year.

**491. A Comparison between Machine-and Hand-mixed Concretes.—**If a sufficient number of turns is employed, concrete can be as well mixed by hand as by machine. However, the cost of such thorough work pro-

\* See *Jour. Western Soc. of Engr.*, Vol. 19, p. 813.

† *Engr. Contracting*, Vol. 43, p. 78.

hibits its use in practice. With the practiced methods of hand mixing it is not possible to secure so homogeneous mixtures as with machine mixing. This non-homogeneity of the hand-mixed material is especially noticeable when the permeability of the concrete is tested.

Table 6 shows the results of tests on the compressive strength of 10×24-in. cylinders made of hand- or machine-mixed concrete. The materials were a crushed limestone passing a 1½-in. and retained on a ¼-in. sieve, a good bank sand and a Portland cement of standard quality. Specimens were sprinkled twice a day until tested. In making the hand-mixed specimens the cement and sand were turned twice dry; the cement, sand and stone, twice dry; the whole mass was then wetted and given three more turns. The machine-mixed concrete was turned for one minute dry and two minutes wet in a No. 0 Smith batch mixer running at 26 r.p.m. A medium consistency was used in making all concrete, but the hand mixed required more water for this consistency than the machine mixed.

**492. Handling of Concrete.**—From the standpoint of securing good concrete after placement, certain fundamental principles should be borne in mind in handling the concrete.

For conveying concrete, wheelbarrows and two-wheel carts are commonly employed on small jobs, while elevators with gravity chutes, cars, belt conveyors, and cableways are used on large structures. For lining tunnels, concrete has been transported by compressed air. In using wheelbarrows or carts attention should be given to minimizing the length of haul not only for the sake of economy in labor but to prevent separation of the ingredients in the wheelbarrow due to jarring. The spouting of concrete by gravity requires a very wet, soupy mixture, unless a high pitch is employed; consequently the strength and density of the product suffer when this method is used. Whatever system is employed, the number of changes from one conveying vessel to another should be

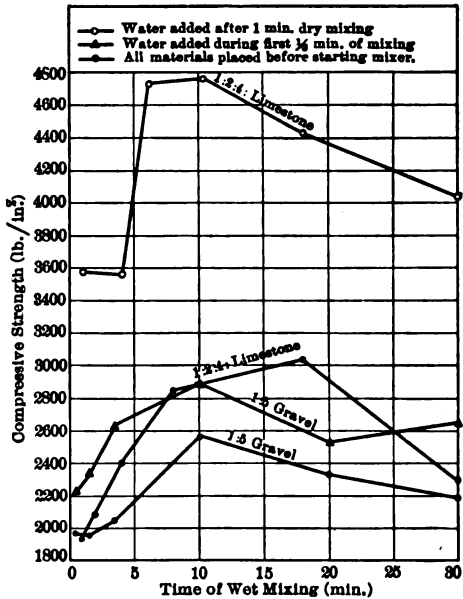


FIG. 10.—Effect of Time of Mixing on the Compressive Strength of 6-inch Concrete Cubes Mixed in a Chicago Improved Cube Mixer (Scofield).

reduced as much as possible in order to avoid waste of mortar due to slopping.

TABLE 6.—A COMPARISON OF THE COMPRESSIVE STRENGTHS OF MACHINE AND HAND MIXED CONCRETES

Proportions (by Vol.).	Method of Mixing.	Age (Days).	Per Cent * Water (by Wt.).	Average Com- pressive Strength † (Lb./in. <sup>2</sup> ).	Ratio of Strengths $\frac{M}{H}$ .	Average $\frac{M}{H}$ .
1 : 1 : 2....	M	28	9.4	3821	1.36	1.38
1 : 1 : 2....	H	28	11.1	2807	.....	
1 : 1 : 2....	M	62	9.4	4650	1.40	
1 : 1 : 2....	H	62	11.1	3327	.....	
1 : 2 : 4....	M	27	9.2	2068	1.34	1.25
1 : 2 : 4....	H	27	9.6	1542	.....	
1 : 2 : 4....	M	58	9.2	2337	1.15	
1 : 2 : 4....	H	59	9.6	2030	.....	
1 : 3 : 6....	M	27	8.7	1268	1.25	1.33
1 : 3 : 6....	H	27	9.1	1015	.....	
1 : 3 : 6....	M	58	8.7	1727	1.42	
1 : 3 : 6....	H	58	9.1	1218	.....	
1 : 4 : 8....	M	26	9.3	645	0.99	1.07
1 : 4 : 8....	H	26	9.6	652	.....	
1 : 4 : 8....	M	58	9.3	1008	1.15	
1 : 4 : 8....	H	58	9.6	879	.....	

\* Does not include moisture in sand and stone which were "air dried."

† Each result is the average of three tests.

For placing concrete under water, some method must be used which prohibits the separation of the cement or mortar from the stone. This is sure to occur if the concrete is poured into open water; the stone, sand, and cement will be found in layers one above the other in the order named. Wherever possible the work should be enclosed in a coffer-dam to avoid wave action and prevent currents about the structure. Enclosed buckets holding  $\frac{1}{2}$  cu. yd. or more, which can be lowered to the bottom and emptied without the contents being subjected to the wash of the water, have been successfully used to deposit concrete to depths of 40 ft. In this method a dry consistency is employed and care is taken to create as little disturbance as possible in raising and lowering the bucket.

Another device which has been reported satisfactory for depositing concrete under water is a tremie or pipe with a conical frustum at the upper end. This tube is filled with concrete before lowering into place. It is then kept full of concrete and gradually shifted over the work by a crane. A report describing the successful use of this method on the Detroit River

Tunnel may be found in *Engr. News*, Vol. 63, p. 420. Tests of 5-in. cylindrical cores cut from the outside of the tunnel when two years old gave a range in compressive strength from 1451 to 4060 lb. per square inch, the average for six test-pieces being 2663 lb. per square inch. Air-cured concrete of the same mix, 1 : 3 : 6, had an average strength of 2097 lb. per square inch.

When there is considerable current about the work, concrete may be deposited in partially filled, loosely woven cloth sacks. The current washes more or less of the mortar through the sacks into the spaces between them, thus bonding the separate units.

**493. Placement of Mortar and Concrete.**—Mixtures of dry consistency should be placed in layers not over 8 in. thick. Wet concrete can be placed in much thicker layers depending upon the consistency and width of the cross-section. Dry concrete should not be allowed to fall more than a few feet, since the coarse aggregate is liable to become separated from the mortar. When concrete must be poured from a considerable height it is desirable to use an inclined chute both to avoid separation of ingredients and to avoid excessive pressure on forms.\*

With mushy or dry concrete the tamping irons shown in Fig. 11 will be found useful in making a dense mix. Smooth exterior surfaces can be produced on vertical concrete walls by running a spading tool up and down next to the forms. Puddling wet concrete in deep, thin walls or columns with long rods is an effective way of avoiding pockets. Pounding the outside of the forms tends to eliminate voids in vertical faces of walls or columns.

In constructing two coarse floors, sidewalks or pavements the base should never be allowed to set before the wearing surface is placed. For the best results, the mortar for the finish coat should be gauged with the minimum amount of water which will admit leveling with a screen. After placement the mortar should not be allowed to partially set but

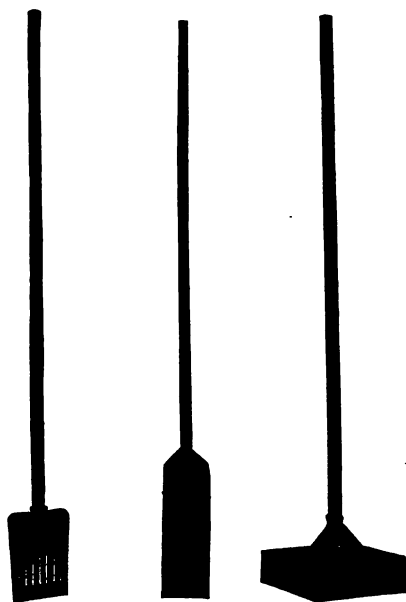


FIG. 11.—Facing and Tamping Irons.  
(a) Gridiron tamper for facing side walls. (b) Flat-faced tamper. (c) Grid tamper for floors or walks.

\* See discussion in *Eng. Record*, Vol. 59, p. 279.

should be screened and finished with a wooden float. Opinions differ concerning the methods of surfacing the work. Specifications of the N. A. C. U.\* specify that the surface shall be finished before the mix has begun to set, that excessive floating is to be avoided and excess water must be removed by mops. Others claim that the surfaces should be vigorously troweled and that whenever excess water appears it should be absorbed by sprinkling a dry mixture of 1 : 1 mortar upon the wet spots. The dry mortar must then be thoroughly floated until the surface stiffens and drags under the float.†

*The Cement Gun.*—An apparatus which is being considerably used for applying a coat of mortar or grout to large surface is the cement gun. The essential feature of this machine is a vertical tank into which the mixture of sand and cement is admitted through a bell hopper. At the lower end of the tank is placed a horizontal wheel which is provided with radial arms to sweep the dry mortar under an outlet pipe. The dry mortar is driven out of the tank by means of air pressure which also serves to operate an air motor that turns the distributing wheel. From the outlet the mix is driven through a flexible hose, which may be of any length up to 200 ft., to a nozzle. As the mixture is shot through the latter it is tempered with sufficient water to produce a plastic mortar and is then deposited at high velocity in the form of a spray on the surface of the work. To facilitate moving, the machine is mounted on a two-wheeled truck.

Tensile tests of 1 : 4 gunite, as the mortar from the cement gun is called, show that a product having higher density and strength than hand-molded mortars of like proportions can be secured.‡ The cement gun has been used for facing the upstream face of a large dam, for reinforcing old sewers, for placing a protective coating of mortar around structural steel in buildings and tunnels, for lining tunnels, and for repairing the walls of furnaces and coke ovens.

**494. Joining Old and New Work.**—Whenever water-tight concrete is desired pouring should be carried on continuously § or a water-tight joint must be provided, see Fig. 12. On less important structures the surface of the old work should be roughened thoroughly, cleaned of all laitance and dirt with wire brushes, saturated with water, and then given a thin coat of neat cement grout before the new concrete or mortar is placed. Tests on 1 : 2 : 4 concrete cylinders 6 in. in diameter show that the full tensile strength of the concrete can be developed by this method

\* *Proc. N.A.C.U.*, Vol. 7, p. 649.

† See *Concrete Cement Age*, Vol. 8, p. 125.

‡ See *Concrete*, Vol. 9, p. 26.

§ For a good example of what may be accomplished in this way in erecting large standpipes see *Concrete Cement Age*, Dec., 1913 and Feb., 1914.

of bonding a joint. For roughening the surface of the old work a dilute solution of hydrochloric acid is effective.

The results of experiments by E. P. Goodrich \* on the tensile strength of bonded joints in mortar briquettes are given in Table 7. Tests of a similar nature made at Case School of Applied Science, by R. B. Perry,† have been summarized in Table 8.

TABLE 7. BOND TESTS ON PORTIONS OF 1 : 2 STANDARD SAND MORTAR BRIQUETTES. (GOODRICH)

All tests made at thirty days. Average strength of seven whole briquettes = 377 lb. per sq. in.

CHARACTER OF SURFACES BONDED.	ROUGH FRACTURE				SURFACE MOLDED SMOOTH.			
	24 Hours.		7 Days.		24 Hours.		7 Days.	
	Each.	Average.	Each.	Average.	Each.	Average.	Each.	Average.
Age when Bonded.								
How Treated.	Strength in Lb. per Sq. In.							
(a) Air cured . . . .	161	.....	285	.....	162	.....	C	
	C	.....	313	.....	205	.....	H	
	92	126	343	314	80	149	15	15
(b) Soaked . . . . .	H	.....	138	.....	165	.....	H	
	H	.....	218	.....	90	.....	C	
	60	60	212	189	208	154	44	44
(c) Grouted . . . . .	380	.....	347	.....	98	.....	H	
	315	.....	280	.....	43	.....	H	
	C	347	200	276	160	100	H	
(d) Hydrochloric acid . . . . .	68	.....	126	.....	87	.....	83	
	C	.....	175	.....	232	.....	51	
	155	112	221	174	105	141	44	53
(e) " Bondsit " . . . .	30	.....	152	.....	79	.....	172	
	60	.....	105	.....	54	.....	26	
	H	45	95	117	68	67	114	104

H = broken in handling before testing.

C = broken placing in clips of machine.

(a) Half briquettes or fresh mortar were molded against halves of air-cured specimens.

(b) The old portions were thoroughly soaked in water before the new portions were molded against them.

(c) The old portions were dipped in a creamy grout of neat cement before the new portions were molded.

(d) The ends of the old portions were washed with a 10-per cent solution of commercial hydrochloric acid, washed in water and then treated as in (a).

(e) A patented powder, " Bondsit," was dissolved in water (5 lb. to 10 gal.) and used as in (d).

\* *Eng. News*, Vol. 61, p. 321.

† *Eng. News*, Vol. 60, p. 167.

TABLE 8.—A COMPARISON OF THE MODULI OF RUPTURE OF 1:2 MORTAR PRISMS BONDED IN VARIOUS WAYS. (PERRY)

Prisms  $2\frac{1}{2} \times 2\frac{1}{2} \times 13\frac{1}{2}$  in. were molded end to end with similar prisms fourteen days old. Twenty-one days after bonding each pair of prisms was subjected to a uniform bending moment and broken (Feret's method).

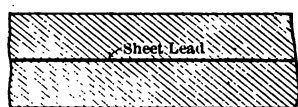
No.	Character of Joint.	TENSION IN OUTER FIBER (LB./IN. <sup>2</sup> )		Per Cent of Full Strength Developed.
		Individual.	Average.	
A1.....	End roughened with a cold chisel and wet.	158	124	49
A2.....		123		
A3.....		87		
A4.....		124		
A5.....		128		
B1.....	End smooth and treated with neat cement grout.	119	133	53
B2.....		131		
B3.....		207		
B4.....		82		
B5.....		124		
D1.....	End roughened as in A and treated with neat cement grout.	279	236	94
D2.....		211		
D3.....		237		
D4.....		225		
D5.....		227		
E1.....	End smooth and treated with "Ransomite." *	232	191	76
E2.....		128		
E3.....		220		
E4.....		231		
E5.....		146		
F1.....	End previously prepared by being molded with a bonding groove.	173	133	53
F2.....		133		
F3.....		120		
F4.....		128		
F5.....		110		
G1.....	Full-length prisms, no joint.	255	252	
G2.....		230		
G3.....		249		
G4.....		271		
G5.....		257		

\* A patented liquid compound which is used to cleanse and roughen the surface of the joint.

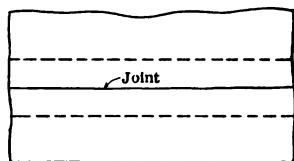
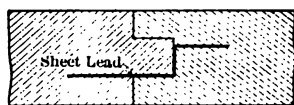
**495. Forms.**—To prevent leakage of water and a loss of fine mortar, the forms should be made as rigid and nearly water-tight as possible. This

feature should be given very careful attention when thin, water-tight sections are being constructed. If smooth surfaces are desired tongued and grooved lumber planed on one side should be used. Oiling the forms diminishes warping and shrinkage and reduces labor in removing them. Collapsible steel molds or wooden forms covered with galvanized iron are economical for repeated use.

The pressure of unset wet concrete is equivalent to a liquid weighing approximately 150 lb. per cubic foot.\* Therefore forms filled before setting begins should be designed for such hydrostatic pressure. After the concrete has begun to set the pressure on the forms decreases. The experiments of F. R. Shunk \* Fig. 13, show the maximum pressures obtained on forms under various rates of filling in different temperatures. The pressures were determined on a board 9.23 in. in

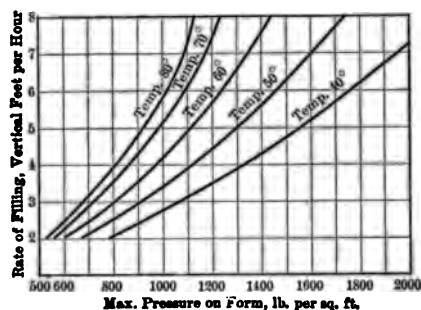


SECTION AT JOINT

ELEVATION  
(a) JOINT HORIZONTAL

(b) JOINT VERTICAL

FIG. 12.—Methods of Making Watertight Joints.

FIG. 13.—Relation of Pressure Exerted by Fluid Concrete on Forms to Rate of Filling (*Engr. News*, Vol. 62, p. 288.)

diameter placed near the bottom of a side form supporting a heavy wall. A wet consistency of  $1 : 3 : 5\frac{1}{2}$  concrete, into which the workmen sank to a depth of  $1\frac{1}{2}$  ft. was used.

Shunk's values have been considered too large,† but later tests by E. B. Germain ‡ corroborate the pressures for wet concrete very rapidly poured. Germain recorded the hydrostatic pressures developed in hot-water bags embedded at the bottom of 18 ft. columns. Mixes of  $1 : 1\frac{1}{2} : 3$  and  $1 : 1 : 1$  concrete were employed. The column made from the first mix was poured in 9 min., during which time the weight per cubic foot of the equivalent fluid decreased from 169 to 140 lb. The other column composed of the

\* *Eng. News*, Vol. 62, p. 288.† *Ibid*, Vol. 63, p. 748; Vol. 64, p. 103.‡ *Ibid*, Vol. 70, p. 294.



second mix was poured in 14 min., the accompanying pressure change corresponding to a decrease in weight per cubic foot from 148 to 138 lb.\*

The time at which forms may be removed is dependent upon the rate of hardening of the cement and the temperature of the air. The best index of the proper time for removal of forms is afforded by a series of tests on small beams or cubes made and cured under the same conditions as the structure. Forms should never be removed until the concrete will support 150 per cent of its working unit stress. In warm weather, wall forms not over 10 ft. high can be removed in two or three days, but in the spring and fall when the temperature at night drops to 30° or 40° F., four to seven days should be allowed, see also Art. 539.

**496. Shrinkage in Setting.**—Owing to the shrinkage which takes place when concrete sets in air, see Art. 357 and 522, and the volume changes which occur due to variations in temperature, provision for contraction and expansion joints should be made.

In two-course sidewalks it is good practice to make joints  $\frac{1}{8}$  to  $\frac{1}{4}$  in. wide, every 4 to 6 ft. Such joints should extend through both surface coat and base. In concrete pavement constructions and in unreinforced walls, joints are placed from 30 to 50 ft. apart.†

The width of the joint now used is quite variable. Some engineers advocate simply a plane of separation others make the joint  $\frac{1}{4}$  to  $\frac{1}{2}$  in. wide and fill with tar or a prepared fiber board. In thin members where the moisture changes are felt throughout the mass, wider joints will be more necessary than in thick sections.

The tongued and grooved joint shown in Fig. 13 is impervious and also serves to preserve alignment when used in a vertical wall. A strip of sheet lead, bent as shown, permits free articulation of the joint. Provision for such joints should be made at all angles in order to avoid cracks due to settlement, contraction or expansion.

**497. Curing.**—No part of the process of making good mortar or concrete is more important than thorough curing. It is also one of the operations most frequently neglected. Dusty floors, loose surface coats on sidewalks and pavements, weak concrete blocks, leaky conduits and pipes illustrate defects frequently caused by improper curing. The effects of premature drying on the strength and permeability of concrete are discussed in Art. 529.

In warm weather the essential principle is to keep the work damp for a period of two to four weeks subsequent to pouring. Rich mixes do not require so long time for curing as lean mixes and are less affected by pre-

\* Further evidence corroborating the work of Shunk and Germain is furnished by tests of Prof. A. B. McDaniel and N. B. Garver, *Eng. News*, Vol. 75, p. 933.

† See Specifications for Concrete Roads, Streets and Alleys adopted by Second National Conference on Concrete Road Building.

mature drying. Wet mixes suffer less than those of dry consistency when improperly cured. If provision cannot be made for wetting the work, the forms should be left on for three or four weeks.

The following methods of curing yield good results, sprinkling two or three times a day when not exposed to the sun, covering with a canvas wetted twice a day, covering with a 2-in. layer of damp sand, earth or sawdust wetted once a day, impounding a shallow pool of water over the surface of the work. Of these methods the wet sand treatment is very effective for pavements and floors and is applicable to a wide range of conditions.

For curing cement blocks and tile, specifications recommending a steam curing period of forty-eight to ninety-six hours subsequent to initial set have been proposed.\* In warm weather the shorter steaming period is recommended followed by sprinkling three times a day for at least one week. Since the object of steam curing is to accelerate hardening the steam should be wet and the temperature of the curing rooms should be kept above 70° F. Wet steam can readily be secured by admitting it to the curing chambers through troughs of water placed on the floor.

**498. Protection against Freezing.**—In cold weather, concrete should be protected from freezing until it has secured hard set. The effects of freezing on the properties of cements, mortars and concretes are discussed in Arts. 359, 539, and 541. Some of the ways of preventing freezing will be briefly considered.

Tests on small 8-in. walls poured in 2-in. plank forms at a temperature of 10° F. have shown that concrete will set before freezing begins provided the temperature of the concrete is above 70° F. when it is poured. This temperature may generally be maintained by heating the mixing water alone. If necessary the aggregates may be heated by building fires in large iron pipes running through the piles of sand and stone. Dolomitic and calcareous sands, however, may be injured by overheating in this manner. When steam can be had, radiators may be employed or the free steam can be piped into the bottom of the material pile. Since the sand is wetted by use of the latter method, a longer mixing period will be required when it is used.

After placement the concrete can be kept warm by covering it with a couple of feet of straw or hay, or a heavy layer of sawdust may be employed. Since these materials get mixed with the surface to some extent their use is often objectionable.

For heating buildings in the process of construction salamanders or box-stoves are often used. Exposed walls and floors have been heated by placing a covering of canvas or building paper a few inches from the sur-

\* *Proc. N.A.C.U.*, Vol. 7, pp 764 and 768; see also Vol. 6, p. 615; Vol. 7, pp. 770 and 789.

face of the concrete and running steam pipes between the covering and work. Precaution should be taken when a structure is being heated to keep the air saturated with moisture in order to prevent too rapid drying of the hardening concrete.

Tempering the mixing water with salt or calcium chloride solutions to lower the freezing-point of concrete is a practice to be condemned, especially if the work is reinforced. Either of these ingredients weaken the concrete and decrease the resistance of reinforced concrete to corrosion. (See Art. 549.)

## CHAPTER XIV

### THE PHYSICAL PROPERTIES OF MORTAR AND CONCRETE

**499. Introduction.**—In this chapter we shall consider the effects of various elements and conditions which greatly influence the properties of mortar and concrete, such as strength and elastic properties, permeability to water, absorption, thermal properties, and the durability. In most cases the results given are from laboratory experiments; and it should be kept in mind that only by exercising the utmost care in selecting, proportioning and mixing materials and in the placement and curing of the concrete will it be possible to secure similar results under the conditions of practice. Also it must be recognized that many of the results represent only a limited range of variables and deductions should not be made for conditions lying without this range.

#### STRENGTH OF MORTARS

**500. Effect of Proportion of Cement on Mortars.**—The results of Feret's \* tests on mortars made from fine, medium and coarse sands (Fig. 1), show, in a general way, the effect of the proportion of cement on strength. Each tension value is the average from 25 briquettes; each compression result represents five cubes 2.8 in. on a side; each transverse value was averaged from tests on 15 prisms, 0.8×0.8 in. in cross-section loaded at the center of a 3.9 in. span; and each shear test represents the average obtained from 15 halves of the transverse specimens. The latter were tested as cantilevers with the load applied close to the support. All mixes were of plastic consistency. The test-pieces were cured in water for five months before testing.

The influence of age upon the strength of water-cured mortars made from several of Wisconsin † sands and screenings is illustrated in Fig. 2. Information concerning the aggregates used in these mortars is given in Table 4, Ch. XIII.

**501. Effect of Character of Fine Aggregate on Mortars.**—From a large number of tests by the United States Geol. Survey (*Bulletin* No. 331), the results in Table 1 have been drawn. Mechanical analysis diagrams for

\* *Bulletin de la Société d'Encouragement pour l'Industrie Nationale*, 1897, p. 1593; article by R. Feret, Chef du laboratoire des Ponts et Chaussées.

† *Proc. A. S. T. M.*, Vol. 13, p. 834.

several of these fine aggregates may be found in Fig. 5, Ch. XIII. These tests show in a rough way that the density and strength of mortars made of the same class of aggregates decrease as the proportion of fine grains in the aggregate increases. In strength, the mortars made of stone screenings are slightly superior to sand mortars.

From a series of tests on approximately 115 natural sands which were reported in *Technologic Paper No. 58*,\* the relation between density ( $\rho$ )

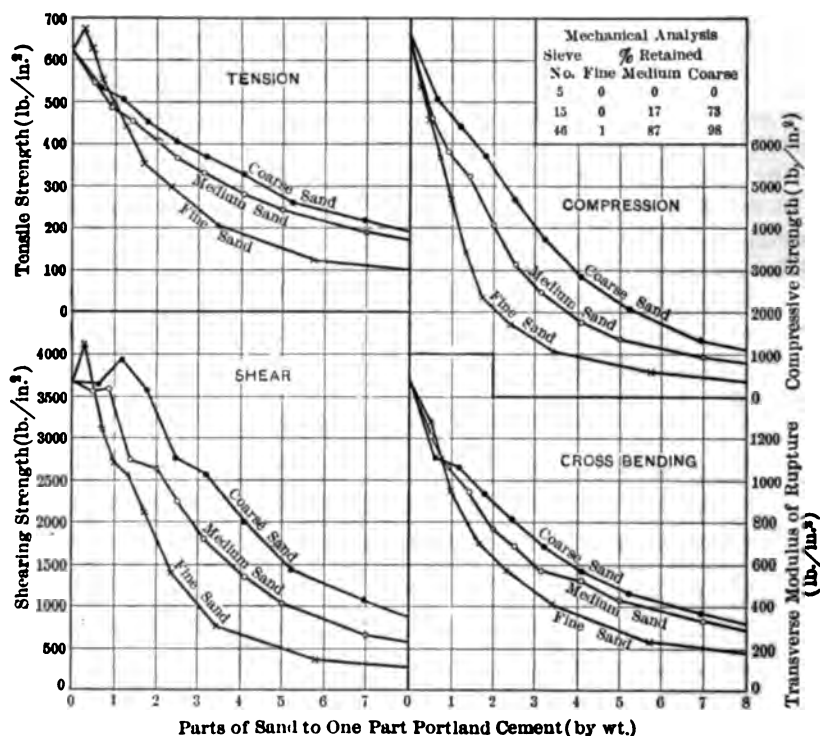


FIG. 1.—Results of Strength Tests on Portland Cement Mortars of Different Proportions Made from Fine, Medium, and Coarse Sands (Feret).

and the average compressive strength ( $S_c$ ) of 2-in. cubes of 1:3 mortar was  $S_c = 26500 \rho - 14750$ . Practically all of the strengths were within 30 per cent of the values given by this equation. In these tests a mixture of several brands of cement, a plastic consistency, hand tamping with moderate pressure, and water-storage were used. Variables in these factors as well as the character of the grains of fine aggregate will affect the constants in equations like the above.

\* By Wig, Williams and Gates of the U. S. Bureau of Stds.

**502. Experiments on Mortars with Artificially Graded Sands.—**  
 One of the most exhaustive researches on the effects of granulometric com-

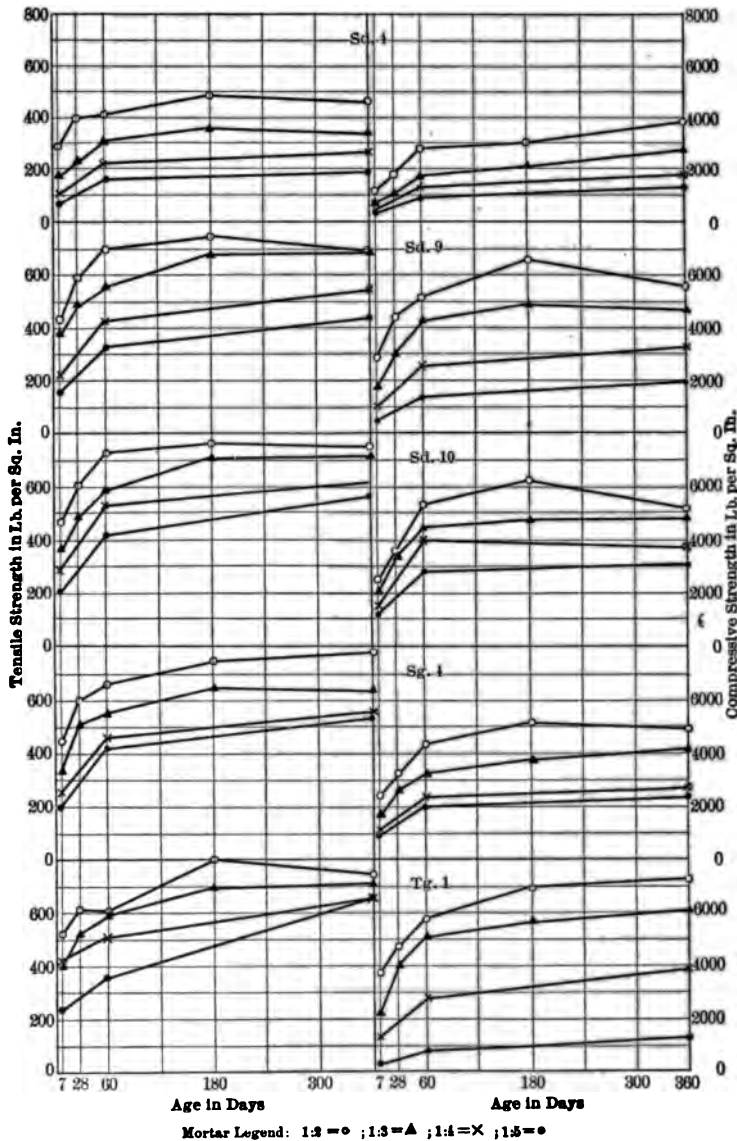


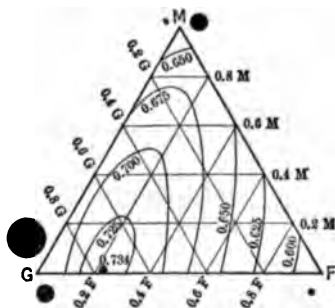
FIG. 2.—The Effect of Age on the Strengths of Water-cured Mortars Made with Portland Cement. (Each point represents three or four tests.)

position of sands has also been conducted by M. Feret. In an important series of tests he used sands graded as follows: Large grains *G* (0.2 to 0.08

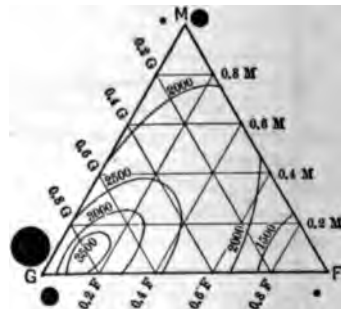
**TABLE 1.—EFFECT OF GRADATION OF SIZES OF PARTICLES OF AGGREGATE ON STRENGTH, DENSITY, AND YIELD OF 1 : 3 MORTARS OF PORTLAND CEMENT**(U. S. Geol. Sur., *Bull.* No. 331)

Aggregate No.	PER CENT RETAINED ON SIEVE NO.				Per Cent Voids.	Yield.	Density.	STRENGTH AT 180 DAYS IN LB. PER SQ. IN.		
	10	30	50	100				Ten-sile.	Com-pressive.	Trans-verse.
Sd. 13.....	31	77	93	99	28.9	1.21	.754	668	7183	1314
Sd. 16.....	40	79	95	99	29.7	1.20	.760	605	7108	984
Sd. 19.....	27	70	92	98	26.9	1.19	.789	773	6719	
Sd. 20.....	23	64	83	97	28.0	1.16	.794	670	6200	
Sd. 11.....	32	63	80	94	36.0	1.13	.730	708	5067	1272
Sd. 10.....	6	33	73	95	31.6	1.18	.743	488	4639	
Sd. 1.....	3	43	79	84	32.5	1.18	.742	415	3677	1056
Sd. 21.....	1	17	80	99	40.9	1.05	.700	380	2892	
Sd. 15.....	1	5	39	93	40.5	1.13	.676	331	2633	792
Sg. 4.....	74	85	90	94	36.0	1.08	.755	939	8644	1602
Sg. 6.....	68	82	86	88	33.1	1.11	.760	750	8048	1326
Sg. 10.....	50	78	88	92	31.8	1.11	.763	767	7394	1218
Sg. 3.....	35	57	73	85	37.0	1.09	.733	809	6500	1410
Sg. 12.....	31	66	79	87	35.1	1.13	.733	717	6193	1446
Sg. 7.....	47	82	91	94	36.1	1.07	.756	677	5279	1206
Sg. 21.....	6	28	47	70	41.0	1.14	.655	683	3948	
Sg. 11.....	10	69	54	80	42.1	1.12	.709	543	3757	918

in.), medium grains *M* (0.08 to 0.02 in.), and fine grains *F* (0.02 to 0 in.). In analyzing results he plotted points representing the proportions of the



**FIG. 3.—The Effect of the Gradation of the Sizes of Sand Grains on the Densities of Mortars of 1 : 3 Proportions (by weight).** (The range in actual size of grain corresponding to a given letter is represented by the pair of circles at each apex in the diagram.)



**FIG. 4.—The Effect of the Gradation of the Sizes of Sand Grains on the Compressive Strength of 1 : 3 Portland Cement Mortars after Storing One Year in Fresh Water.**

three sizes on equilateral triangular diagrams, the distance from any side representing the percentage of grains marked at the opposite vertex. The contour lines were then drawn through the points which corresponded to mortars of the same density, strength, etc. A comparison of Figs. 3 and 4 shows the similarity of the contours in the density and strength diagrams of 1 : 3 mortars and indicates that the maximum strength and density obtain when about five-sixths of the sand is composed of the coarse grains with fine grains constituting the principle portion of the remainder. Other tests \* by Feret furnished the data for Fig. 5. From the experiments, Feret drew the following conclusions:

1. With cement varying between 10 and 30 per cent of the weight of sand, the strongest mortar for any given percentage of cement was always gotten from a proportion of coarse sand equal to twice the weight of the cement plus fine sand.

2. It requires about twice as much cement mixed with a given quantity of sand to produce a mortar of given strength when fine sand is used as it does with coarse sand.

3. The weight of cement per cubic yard of mortar of a given strength is about twice as much for fine sand as for coarse sand, with the ordinary mixtures.

4. The cost per cubic yard of coarse-sand mortar of a given strength (such as is found for the ordinary ratio 1 c. : 3 s.), is only about 75 per cent of the cost of fine-sand mortar of the same strength, even when the coarse sand costs six and one-half times as much as the fine sand (coarse sand \$1.30, and the fine sand \$0.20 per cubic yard).

Feret also declared † that "for all series of plastic mortars made with the same cement and of inert sands, the resistance to compression after the same time of set under identical conditions is solely a function of the ratio  $\frac{c}{e+v}$  or  $\frac{c}{1-(c+s)}$ , whatever may be the nature and size of the sand the proportions of the elements—sand, cement, and water—of which each is composed." In the above law  $e$  and  $v$  represent the volume of the water and air voids, respectively; the other symbols are defined in Art. 464. He derived the following relations for compressive strength:

$$S_c = j \left[ \frac{c}{1-(c+s)} - 0.1 \right], \text{ and } S_c = k \left( \frac{c}{1-s} \right)^2,$$

in which  $S_c$  = unit compressive strength in pounds per square inch and  $j$  and  $k$  are constants. His results indicated a value of  $k$  = 26,000 lb. per

\* *Annales des Ponts et Chaussées*, Mar., 1890, July, 1892, Aug., 1896.

† *Bull. de la Soc. d'Encouragement l'Ind. Nat.*, 1897, p. 1604.



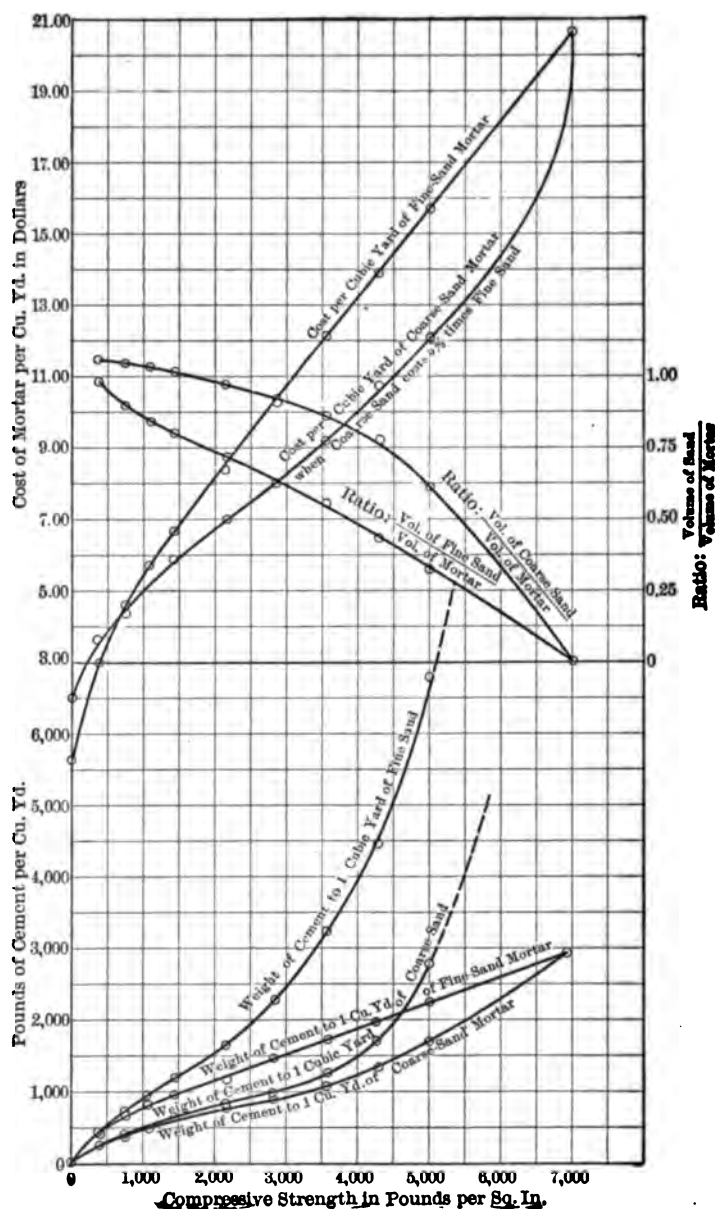


FIG. 5.—Relative Economy of Coarse and Fine Sand in Portland Cement Mortar after Five Months' Immersion in Sea Water. Coarse sand grains were graded as follows: 52 per cent passing No. 5 and held on No. 15, 48 per cent passing No. 15 and held on No. 46; fine sand grains, 25 per cent passing No. 15 and held on No. 46, 75 per cent passing No. 46 sieve. (M. Feret in *Les Matériaux de Construction* (Baumaterialien-Kunde, Vol. 1, p. 141.)

square inch. These equations with modified constants have been found to hold in other tests.\*

**503. Effect of Proportion of Mixing Water on Strength of Mortars.—**

In general, increasing the percentage of mixing water beyond that required to form a mixture of standard consistency lowers the density and strength. The decrease in strength is most marked, however, at early ages.

Experiments by Reinke † show that the more water required to produce a given consistency the less will be the strength. He found that the ratios of the average strengths of eighteen 1 : 3 mortars taking 12 per cent water for normal consistency to the strengths of twelve like mortars requiring 14 to 15 per cent were 3.9 : 1, 1.6 : 1, and 1.5 : 1 at three, seven and twenty-eight days respectively. Therefore, it follows that the percentage of water required to make a mix of normal consistency is a rough index of the strength of mortars made of sands free from impurities.

**504. Effect of Mica on Strength of Mortar.—**Experiments by W. A.

Willis ‡ on 1 : 3 mortars show that small percentages of mica decrease the tensile strength considerably. An addition of 2½ per cent of mica served to reduce the strength at seven days 11 per cent; by adding 20 per cent of mica the strength at the same age was decreased from 180 to 40 lb. per square inch. Feret also found that mica adversely effected the compressive strength. The observed effects are probably due to the weakness of the mica and to the decrease in density resulting from its presence.

**505. Effect of Hydrated Lime on Strength of Mortars.—**Tests by

E. W. Lazell, § Table 2, show that replacement of less than 15 per cent of the cement by hydrated lime does not decrease the tensile strength of 1 : 3 mortars. The results of E. S. Wheeler (*Report of Chief Engineer, U. S. A.*, 1896, p. 2823) are confirmatory. W. E. Emley and S. E. Young ¶ found that both tensile and compressive strengths of 1 : 3 mortars of slush consistency were adversely affected by the substitution of only 5 per cent of either high-calcium or dolomitic hydrate, if the specimens were cured in water or exposed to the weather. However, the loss in compressive strength was small for replacement of less than 25 per cent of cement by hydrate.

**506. Adhesion of Mortars and Concretes.—**Experiments made by

E. Candlot for the French Commission show that the normal adhesion block is a satisfactory form of specimen (see Art. 450), but the tests plainly indicate that the character of the surface and the kind of cement exert a very

\* *Proc. A. S. T. M.*, Vol. 13, p. 852.

† *Proc. A. S. T. M.*, Vol. 13, p. 797.

‡ *Eng. News*, Feb. 6, 1908.

§ *Proc. A. S. T. M.*, Vol. 8, p. 418.

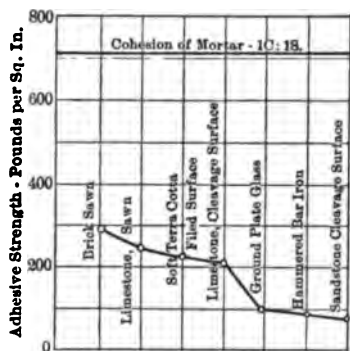
¶ *Proc. A. S. T. M.*, Vol. 14, p. 339.

TABLE 2.—THE EFFECT OF THE INCLUSION OF HYDRATED LIME ON THE TENSILE STRENGTH OF 1 : 3 MORTARS OF PORTLAND CEMENT AND STANDARD SAND. (LAZELL)

Per Cent Cement Replaced by Hyd. Lime.		0	5	10	15	20	25	30	100
Method of Curing.	Age in Days.	Tensile Strengths in Lb. per Sq.In.							
In air, specimens moistened once a week	7	209	203	205	209	133	112	141	10
	28	266	258	255	245	203	170	225	44
	90	286	289	295	297	197	117	177	55
	180	382	312	304	281	229	211	219	53
	270	607	545	441	499	441	397	342	136
	360	630	456	513	642	553	444	327	168
In water after 3 days in air	7	206	157	189	239	237	173	173	
	28	278	311	364	264	268	259	268	
	90	441	389	419	372	374	314	281	
	180	358	321	341	278	260	207	253	
	270	390	301	308	279	268	250	232	
	360	426	336	311	322	299	260	231	

NOTE.—Each value represents five tests.

marked influence upon the adhesion. The tests of Wheeler show that the adhesion of mortar to cut stone is not increased by roughening the surface of the stone. In making adhesion test-pieces Wheeler used small discs, 1 in. square by  $\frac{1}{4}$  in. thick, placed transversely at the center of a briquette mold. These were pulled apart in an ordinary briquette tester. Some of his results on various substances are shown in Fig. 6. For sawn limestone and proportions varying from neat to 1 : 2 he found the adhesion was between 30 and 40 per cent of the tensile strength of the mortar. To secure maximum adhesion the consistency must be considerably softer than standard and the surface of the substance should be thoroughly saturated. Retempering the mortar reduced adhesive strength but, on the other hand, it appears to lessen the shrinkage in setting.\* In curing, rapid

FIG. 6.—Adhesive Strength of Portland Cement Mortar, 1 C. : 1 S., Twenty-eight Days Old, to Different Substances, and the Cohesive Strength of the Mortar Itself. (Wheeler, *Rept. Chf. Engr.*, 1895, p. 3019.)

drying of the mortar should be prohibited.

\* *Cement Concrete Age*, Vol. 13, p. 97.

The adhesion of concrete or mortar to plain round steel bars with surfaces as received from the rolls is about one-eighth to one-tenth of the compressive strength of 6×12-in. cylinders of the same material. For bars with polished surfaces the adhesion is 40 per cent less and for square bars about 25 per cent less than the round rods with mill surface. The adhesion of concrete to flat bars is more variable, but may be considered the same as for square bars.\*

## STRENGTH OF CONCRETE

**507. Effect of Proportion of Cement on the Compressive Strength of Concrete.**—With the density remaining constant and with making and  
**TABLE 3.—THE EFFECT OF PROPORTION OF PORTLAND CEMENT ON THE COMPRESSIVE STRENGTH OF 12-INCH CONCRETE CUBES.**  
 (KIMBALL)

Proportions.	Brand of Cement.	COMPRESSIVE STRENGTH (LB./IN. <sup>2</sup> )			
		7 Days.	1 Month.	3 Months.	6 Months.
1 : 0 : 2...	Alpha.....	3294	5053	5047	
	Germania.....	2734	3246	3858	5129
	Alsen.....	3118	3240	3710	5332
	Average.....	3049	3846	4205	5230
1 : 2 : 4...	Saylor.....	1724	2238	2702	3510
	Atlas.....	1387	2428	2966	3953
	Alpha.....	904	2420	3123	4411
	Germania.....	2219	2642	3082	3643
	Alsen.....	1592	2269	2608	3612
	Average.....	1565	2399	2896	3826
1 : 3 : 6...	Saylor.....	1625	2568	2882	3567
	Atlas.....	1050	1816	2538	3170
	Alpha.....	892	2150	2355	2750
	Germania.....	1550	2174	2486	2930
	Alsen.....	1438	2114	2349	3026
	Average.....	1311	2164	2522	3089
	Saylor.....	675	800	1128	1542
	Atlas.....	594	1090	1201	1583
	Alpha.....	564	1218	1257	1532
	Germania.....	759	987	963	815
	Alsen.....	417	873	844	1323
	Average.....	602	994	1079	1359

NOTE.—For character of aggregate see Table

\* For further information on adhesion and bond see *Bulletin No. 71*, Engr. Expt. Sta., University of Illinois, and *Bulletin No. 321*, University of Wisconsin.

TABLE 4.—PROPERTIES OF AGGREGATES

Abbreviations: Sd = sand; Sg = screenings; Gl = gravel; Le = limestone; Ge = granite;  
Tp = traprock; Ce = conglomerate

Table No.	Fig. No.	Authority.	Kind of Aggregate.	COARSE AGGREGATE		FINE AGGREGATE			
				Range in Size in In.	Per Cent Voids (Loose).	Per Cent Passing			Per Cent Voids (Loose).
						No. 10.	No. 30.	No. 100	
3	..	Kimball.....	Sd + Ce	2½-½	49	....	....	....	33
5	..	Univ. Ill.....	Sd + Le	1½-½	50	81	35	6.9*	28
5	..	Univ. Wis.....	Sd + Le	1½-½	52	67	39	4.0	36
9	7	Univ. Wis.....	Sd + Le	1½-½	48	83	38	3.0	34
6	..	Fuller and	Sg + Ge	2½-½	....	77	56	15.0	
6	..	Thompson	Sd + Gl	2½-½	....	79	50	4.6	
			Sd + Tp	½-½	46	Sand passed ½-in. m.			36
			Sd + Tp	¾-½	50		do		
			Sd + Tp	1-¾	50		do		
7	..	Tests of Metals, 1898	Sd + Tp	1½-1	47		do		
			Sd + Tp	2½-2	50		do		
			Sd + Gl	¾-½	33		do		
			Sd + Gl	1½-1	36		do		
			Sd + Gl	3-2½	39		do		
8			Sd + Ge	1-0.02	41	97	64	1.3	38
and	12	U. S. Geol. Surv.	Sd + Gl	1-0.08	33	97	64	1.3	38
13		Bull. 344	Sd + Le	1-0.02	37	97	64	1.3	38
			Sd + Cr	1½-0	51	97	64	1.3	38
	15	Withey.....	Sd <sub>4</sub>	....	....	100	96	8.6	40
	15	Withey.....	Sd <sub>5</sub>	....	....	82	55	1.9	36
	15	Withey.....	Sd <sub>6</sub>	....	....	67	14	1.4	36
18	..	Withey.....	Sd + Gl†	1½-½	36	83	57	1.6	37
					38	68	38	2.4	34

\* No. 74 mesh.

† Range of values for different lots of Janesville gravel and sand.

curing conditions similar, the strength of concrete increases in nearly direct ratio to the proportion of cement for the mixes commonly used. An illustration of the effect of the proportion of cement upon the strength at various ages is furnished by the comprehensive tests of G. A. Kimball.\* The consistency of the concrete was such that, when it was placed in the 12-in. cube molds, water flushed to the surface under vigorous ramming. The specimens were hand mixed at temperatures near freezing and were stored from two to seven days in a room at a temperature approximating 40° F. Thereafter they were cured in wet earth. The properties of the aggregates used in these tests are given in Table 4 and the results of strength tests in Table 3. In nearly all cases each average represents five or six tests.

\* *Tests of Metals, 1899.*

TABLE 5.—TESTS AT THE UNIVERSITIES OF ILLINOIS AND WISCONSIN ILLUSTRATING THE EFFECT OF PROPORTION OF PORTLAND CEMENT ON THE STRENGTH OF BROKEN STONE CONCRETE OF MEDIUM WET CONSISTENCY

Proportions by Volume.	Average Compressive Strength (Lb./In. <sup>2</sup> )	Age in Months.	Form of Specimen.	No. of Tests.	Per Cent Variation from Mean.	Method of Mixing.	Published in
1 : 1½ : 3	2303	2	12-in. cube	4	18	Hand	Univ. Ill.
1 : 2 : 4	1972	2	"	6	12	"	Bull. No. 20
1 : 3 : 6	1450	2	"	2	1	"	"
1 : 4 : 8	1111	2	"	4	14	"	"
1 : 1½ : 3	3808	10	6-in. cube	9	24	"	"
1 : 2 : 4	3412	10	"	12	26	"	"
1 : 3 : 6	2433	10	"	6	7	"	"
1 : 4 : 8	1632	10	"	9	21	"	"
1 : 1½ : 3½	4433	2	6-in. cyl.	8	20	Machine	Univ. Wis.
1 : 2 : 4	2211	2	18-in. high	14	13	"	Bull. No. 466
1 : 3 : 6	1770	2	"	4	18	"	"

NOTE.—The 12-in. cubes and cylinders were cured in the respective laboratories, being sprinkled twice daily. The 6-in. cubes were cured under damp sand. For aggregate see Table 4.

Similar experiments on both hand- and machine-mixed concrete of wet consistency may be found in Table 6, Art. 491. Other results of tests made at the Universities of Illinois and Wisconsin showing the effect of the proportion of cement on the strength of concrete are shown in Table 5.

**503. The Increase in Strength of Concrete with Age.**—There is a scarcity of reliable data on the compressive strength of concrete more than two years old. For the proportions commonly used, one may expect the following percentages of the two-year strength when the mixing and curing conditions are good: 25 to 40 per cent at seven days, 50 to 65 per cent at one month, and 70 to 90 per cent at six months. Concrete of dry consistency will show a much quicker growth in strength than concrete of wet consistency. A series of experiments on the effect of age on the strength of concrete cylinders cured in three different ways was described in *The Wisconsin Engineer*, Vol. 19, p. 203. The Atlas cement for these tests was mixed and stored in an air-tight tank; the proportions, amount of water, time of machine mixing and storage in molds were each made constant for the entire series of tests. Specimens from each batch of concrete were distributed throughout the various ages so that the five results gotten for any given test period are representative of different batches of concrete. The character of the aggregates may be found in

Table 4 and the results of experiments for a seven-year period are given in Fig. 7.

In Fig. 8 are shown the average results of a large number of compressive strength tests \* on gravel, granite, and limestone concretes of 1 : 2 : 4 and 1 : 3 : 6 proportions. These tests were all conducted under comparable conditions. A plastic consistency was used in all tests. The

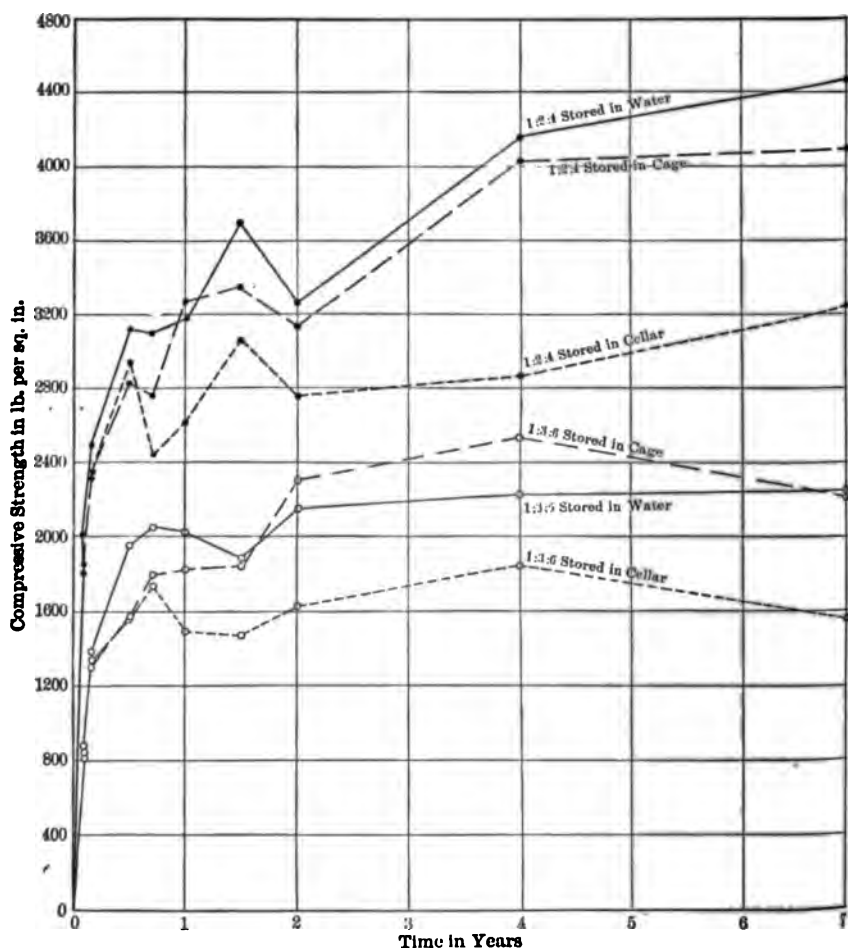


FIG. 7.—The Effect of Age on the Compressive Strength of Limestone Concrete.

figures on the diagrams indicate the number of materials averaged; the number of test-pieces per material was three or more.

#### 5C9. Effect of Density on the Compressive Strength of Concrete.—

If the kind of cement and the proportion of cement per unit volume of

\* From *Technologic Paper No. 58*, U. S. Bureau of Standards.

concrete is maintained constant and if the consistency, shape of aggregate particles, age, and method of curing the concrete are the same, the strength will increase with the density. In proportioning concrete it is rarely possible to keep constant the proportion of cement per unit volume of concrete. Experiments have shown that the strength varies directly as the ratio  $\frac{c}{1-\rho}$ , the equation being  $S_c = j \left( \frac{c}{1-\rho} - n \right)$ . Here  $S_c$  = unit compressive strength,  $c$  = volume of cement grains in a unit volume of con-

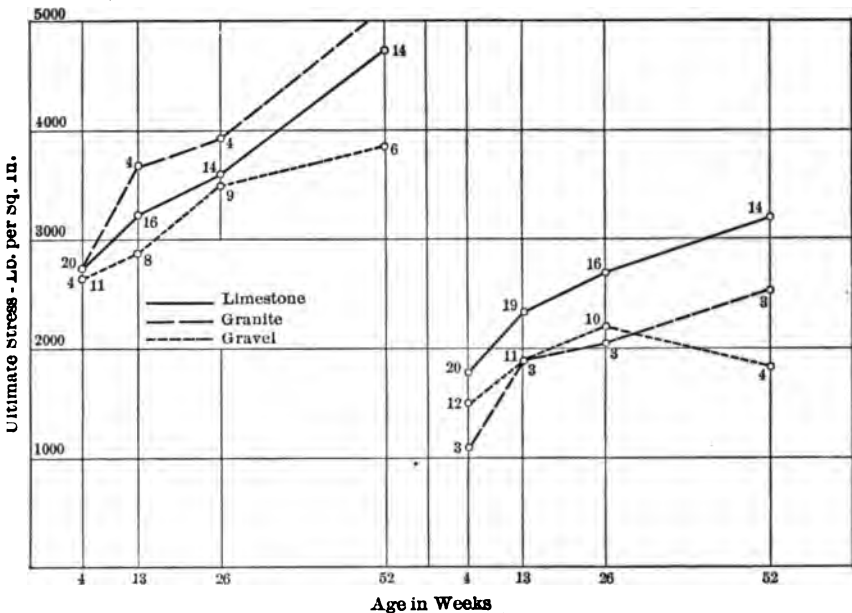


FIG. 8.—Variation of Compressive Strength with Age of 1 : 2 : 4 and 1 : 3 : 6 Concretes in which Granite, Gravel and Limestone Aggregates were Used. (Test Pieces, 8×16-inch Cylinders.)

crete,  $\rho$  = density of concrete, and  $j$  and  $n$  are constants which will vary with the factors mentioned above.\*

Experiments of the Bureau of Standards verify this relation. The points plotted in Fig. 9 represent data from Tables 1, 23, 25 and 26, of *Technologic Paper*, No. 58, by Wig, Williams and Gates. The proportions of the concrete varied between 1 : 6 and 1 : 9 (by volume). Other conditions surrounding the making, curing and testing of the 6-in. cubes used in these tests were constant. Owing to the fact that the specific gravities of the cements used in these tests were not given, a value of 3.10 was as-

\* Further verification of this law may be found in Taylor and Thompson's *Concrete Plain and Reinforced*, 2d ed., p. 367, and in *Jour. West. Soc. Engr.*, Vol. 19, p. 837.



sumed in making computations of  $c$ . In the figure each number represents the average result for three specimens and the like numbers indicate similar aggregates.

**510. Effect of Size of Coarse Aggregate on Compressive Strength.**—With the largest particles not over 3 in. in diameter, it may be stated that the larger the maximum size of the aggregate the denser and stronger will be the concrete, provided other influencing factors are eliminated. The

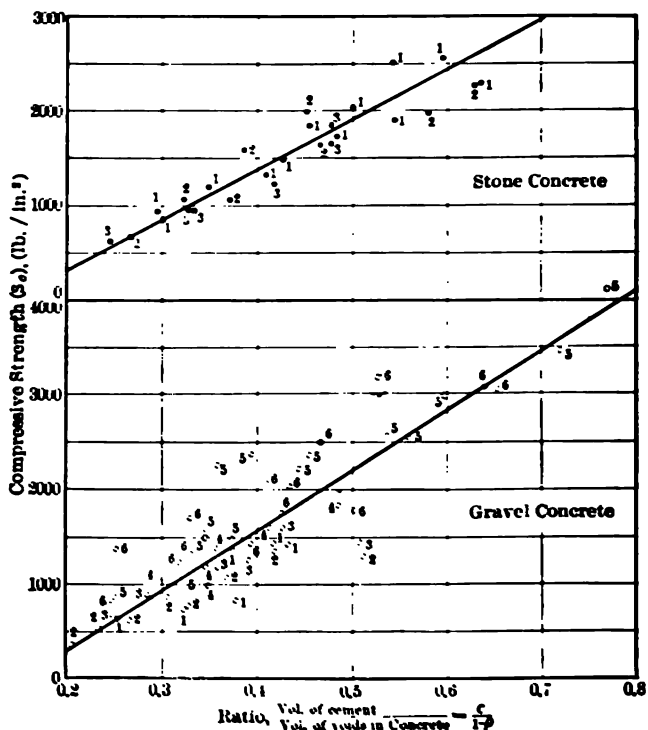


FIG. 9.—The Relation between Compressive Strength and the Ratio  $\frac{c}{1-p}$   
(Age of specimens 4 weeks.)

change in strength with the increase in maximum size of aggregate is most marked for diameters under 1 in. The tests of Fuller and Thompson \* on 1 : 9 concrete, which illustrate the above statement, are shown in Table 6. The beams used in these tests were 6×6×72 in. and the spans were 60 and 30 in. The compression tests were made on prisms from the beams, approximately 6×6×19 in., which were capped on each end with neat cement.

\* *Trans. A. S. C. E.*, Vol. 59, p. 115.

TABLE 6.—EFFECT OF VARIATION IN THE MAXIMUM SIZE OF COARSE AGGREGATE ON THE STRENGTH AND DENSITY OF PORTLAND CEMENT CONCRETE. (FULLER and THOMPSON)

Proportion, by weight.	MATERIAL.*		Character of Mixture.	AVERAGE DENSITY.			AVERAGE MODULUS OF RUPTURE AT 90 DAYS, IN LB. PER SQ. IN.			AVERAGE COMPRESSIVE STRENGTH AT 140 DAYS, IN LB. PER SQ. IN.		
	Stone.	Sand.		2½-in. Stone.	1-in. Stone.	¾-in. Stone.	2½-in. Stone.	1-in. Stone.	¾-in. Stone.	2½-in. Stone.	1-in. Stone.	¾-in. Stone.
1 : 9	{ J. Park	{ J. Park	{ Ideal Natural Uniform Aggregate	0.851	0.810	0.767	257	226	208	1342	950	915
1 : 9	{ "	{ "		0.821	0.798	0.768	211	171	162	980	879	821
1 : 9	{ "	{ "		0.832	0.797	0.769	257	229	180	1350	950	890
1 : 9	{ Cowe* Bay	{ Cowe Bay	{ Various	0.859	0.847	0.853	243	246	189	1486	1402	1231
1 : 9	{ J. Park	{ Cowe Bay		0.872	0.818	0.784	291	273	207	1798	1585	1185
			Averages Ratios	0.847 1.00	0.814 0.96	0.788 0.93	252 1.00	229 0.91	189 0.75	1391 1.00	1153 0.83	1008 0.72

\* Jerome Park stone was a crushed mica schist, the sand was screened from the crusher-run material. Cowe Bay material consisted of gravel and sand.

NOTE.—A soft mushy consistency was used in all tests. For character of aggregates in the natural mixture see Table 4.

The ideal mixture was graded in accordance with the equations of Art. 483.

In Table 7 results from *Tests of Metals*, 1898, show the effect of variations in the size of aggregate on the strength of 1 : 1 : 3 concrete made into 12-in. cubes. These tests also serve as a comparison of the strengths and specific weights for broken stone and gravel concretes.

**511. Effect of Proportion of Water on Strength of Concrete.**—The greatest strength at an early age can be secured from a concrete of dry consistency in which there is only sufficient water for perfect hydration of the cement. Such concrete requires heavy ramming to make it homogeneous and dense. A mushy or quaking consistency is more easily handled and compacted. Although the growth in strength is less rapid, such concrete gains as great strength eventually as concrete of dry consistency. The use of a very wet, sloppy consistency produces concrete of low density and inferior strength.

The above conclusions are well established by Government tests\* on wet, medium, and damp concrete made of granite, limestone and gravel. The results of these tests are given in Table 8. The compression tests were made on 6-in. cubes and the transverse tests on 8×11 in. beams supported on a 12-ft. span. Concrete of wet consistency was sloppy enough to splash when struck with the tamping rod; concrete of medium consistency showed no excess water on surface when compacted in the

\* Bull. 344, U. S. Geol. Survey.

TABLE 7.—EFFECT OF SIZE OF COARSE AGGREGATE ON THE COMPRESSIVE STRENGTH OF PORTLAND CEMENT CONCRETE OF 1 : 1 : 3 PROPORTIONS BY VOLUME (*Tests of Metals*, 1898)

BROKEN TRAP ROCK.					GRAVEL.				
Size of Particles, In.	Age in Days.	Crushing Strength in Lb. per Sq. In.		Average Weight per Cu. Ft., Lb.	Size of Particles, In.	Age in Days.	Crushing Strength in Lb. per Sq. In.		Average Weight per Cu. Ft., Lb.
		Single Tests.	Average.				Single Tests.	Average.	
½-1	7	1391	2137	147.0	¾-1	7	1298	2297	148.1
	19	2220				21	2600		
	32	2800				34	2902		
1-1½	8	1900	2623	148.8	1½-2	7	2276	3093	151.0
	20	2769				22	3186		
	32	3200				29	3817		
1½-2	7	3390	4187	169.3	2-2½	11	2800	3467	151.9
	20	4254				26	3400		
	34	4917				41	4200		
2-2½	11	3189	3919	159.8	3-3½	7	2276	3093	151.0
	26	4006				22	3186		
	41	4562				29	3817		
2½-3	7	2400	3561	159.7	3½-4	11	2800	3467	151.9
	22	4143				26	3400		
	32	4140				41	4200		

NOTE.—For character of aggregate see Table 4.

molds, flowed less smoothly than the wet concrete but could be readily surfaced with a trowel; concrete of dry consistency was very lumpy and granular, under hand ramming it showed no water on the surface, and could not be given a smooth finish with a trowel. After twenty-four hours in the molds specimens were stored in a moist room and sprinkled three times a day.

Tests showing the effect of consistency on both tensile and compressive strength of 1 : 2 : 4 limestone concrete made at the University of Wisconsin are recorded in Table 9. The size, number of results per recorded value, and the curing conditions were the same as those mentioned in the preceding paragraph.

The results of experiments initiated by the *National Asso Cement Users* \* and carried on at different college laboratories are shown in Fig. 10. The tests are noteworthy inasmuch as they were made

\* *Concrete Cement Age*, Vol. 4, p. 141.

TABLE 8.—EFFECT OF PERCENTAGE OF MIXING WATER ON THE STRENGTH OF 1 : 2 : 4 CONCRETE

(U. S. Geol. Sur. Bull. No. 344)

Aggregate.	Percent- age of Water, by Wt.	Consis- tency.	Weight per Cu. Ft. of Cubes, in Lb.	AVERAGE STRENGTH OF THREE SPECIMENS, IN LB. PER SQ. IN.					
				In Compression at			Mod. of Rupture at		
				4 Weeks.	13 Weeks.	26 Weeks.	4 Weeks.	13 Weeks.	26 Weeks.
Granite	8.9	Wet	147.4	3156	4754	4753	375	501	539
	8.3	Medium	147.9	4089	4992	4949	475	536	566
	7.0	Damp	147.7	4518	†5445	†5410	499	591	618
Gravel	9.8	Wet	138.8	2299		3814	391	380	435
	9.0	Medium	142.7	3547		4808	451	477	520
	7.9	Damp	144.8	4612	4989	4884	426	495	496
Limestone	11.0	Wet	144.5	*3072	4008	3460	422	487	507
	10.1	Medium	144.1	2975	3939	3896	458	541	566
	8.5	Damp	147.8	4367	†5451	5025	537	521	589

\* Gotten from tests on 8×16-in. cylinders.

† Exceeded capacity of testing machine; did not fail.

NOTE.—For character of aggregate see Table 4.

TABLE 9.—EFFECT OF PERCENTAGE OF MIXING WATER ON THE TENSILE AND COMPRESSIVE STRENGTH OF 1 : 2 : 4 LIMESTONE CONCRETE

Consistency.	Per Cent Water by Weight.	TENSILE STRENGTH IN LB. PER Sq. IN. at				COMPRESSIVE STRENGTH IN LB. PER SQ. IN. at		
		14 Days.	28 Days.	34 Days.	60 Days.	14 Days.	70 Days.	350 Days.
Dry.....	6	.....	.....	.....	.....	1774	2635	4000
Dry*.....	6½	139	212	206	236	.....	.....	.....
Quaking....	7	.....	.....	.....	.....	1945	3126	4320
Mushy.....	8	180	252	251	272	1709	2927	4500
Soupy.....	10	153	168	204	219	1283	2578	3070

\* Tensile test-pieces of this consistency had air pockets due to difficulties in molding.

NOTE.—For aggregate see Table 4.

ferent parts of the country by different operators using local materials. Identical instructions regarding method of making and testing were sent to all laboratories. All stone employed in these tests passed No. 10 was held on a  $\frac{1}{2}$ -in. sieve. Specimens were 6×6-in. cylinders. Each specimen was hand mixed separately. After removal from the molds, specimens were kept damp until tested.\*

\* For further results illustrating the effect of consistency on the strength of concrete, see C. J. Robinson, *Engr. News*, Vol. 69, p. 1063; also results of Prof. D. A. Abrams, *Engr. News-Rec.* Vol. 80, p. 873.

**512. Tensile Strength of Concrete.**—In general, the tensile strength of concrete varies between one-eighth and one-twelfth of its compressive strength. Besides the affecting conditions discussed under compressive strength tests, imperfections in the fabrication of specimens, the tensile strength and surface characteristics of the aggregate and the method of gripping and loading greatly affect the tensile strength of concrete. Owing to the influence of these factors the results of tension tests are generally less uniform than those gotten in compression.

The results of some of the more important tests, showing both tensile and compressive strengths of concrete, appear in Table 10.

TABLE 10.—A COMPARISON OF THE TENSILE AND COMPRESSIVE STRENGTH OF PORTLAND CEMENT CONCRETE OF MEDIUM CONSISTENCY

Ref. Let.	No. of Tests.		Age of Spec.	Mix by Vol.	STRENGTH (Pounds)		Ratio ( $\frac{S_t}{S_c}$ )	KIND OF AGGREGATE.	
	Tension.	Compr.			Tensile ( $S_t$ )	Compressive ( $S_c$ )		Fine.	Coarse.
H	3	1	1 mo.	1 : 2 : 4	218	3020	13.9	River	Limestone < 1½ in. 46% V.
	2	1	1 m.o.	1 : 2 : 5	203	2097	10.3	Sand	
	4	2	1 mo.	1 : 3 : 6	114	1521	13.3	34% V.	
C	10	4	1 mo.	1 : 2 : 4	207	2534	12.2	L.S. Scr.	Limestone < 1½ in.
	3	4	1 mo.	1 : 2 : 4	161	1662	10.3	Sand	
M	6	3	7 mo.*	1 : 2 : 4	278	2505	9.0	Sand	Limestone < 1½ in.
	6	6	7 mo.	1 : 2 : 4	155	1620	10.4	Sand	Sandstone < 1½ in.
	6	6	7 mo.	1 : 2½ : 5	136	1502	11.0	Sand	< 2½ in.
P	27	37	1 mo.	1 : 2 : 5	237	2290	9.7	Sand	Limestone
			3 mo.	1 : 2 : 5	359	2413	6.7	Sand	< 1 in.
			1 mo.	1 : 5	253	2400	9.5		Pit-run
			3 mo.	1 : 5	290	2804	9.7		Gravel
I	3	2	2 mo.	1 : 3 : 6	169	1600	9.5	Sand	Limestone 1½ in.
W	27	26	1 mo.	1 : 2 : 4	189	1940	10.3	Sand	Limestone < 1½ in.

NOTE.—Specimens H, M, and I were stored in air; P, in dry sand; W, sprinkled; C, in water 9 days then in air.

\* Compression specimens were only one month old.

† Compression specimens were: H—2½×3½×11-in. prisms; C—6×6×14-in. prisms; M—6-in. cubes; P—8×12-in. cylinders; I—8×20-in. cylinders; W—6×18-in. cylinders.

References: H—Henry, *Jour. Assoc. Engr. Soc.*, vol. 25, p. 145. C—Woolson, *Engr. News*, vol. 53, p. 561. M—Mill, *Corsell Civil Engr.*, vol. 19, p. 106. P—Hatt in *Jour. W. S. E.*, vol. 9, p. 234. I—Talbot in *Bull. No. 1*, Univ. of Ill., Expt. Sta. W—Withey in *Bull. No. 197*, Univ. of Wis.

In the tests reported by Talbot and by Withey plain cylindrical test-pieces were used. Both ends of the specimens were fitted with small bolts placed equidistant from and parallel to the axis of the cylinder. In testing, Talbot fastened the small bolts on each end of the specimen to a circular plate which had a hole in the center. A bolt having a spherical head was passed through the hole so that the bolt head bore against the side of the plate nearest to the test-piece. By gripping the shanks of the bolts in the heads of the testing machine an axial pull on the test-piece was approximated. Withey used a crossed knife-edge suspension in place of the bolt with spherical head. Woolson employed prisms of square cross-section with enlarged ends shaped like those on a briquette. He also used special holders to avoid loading specimens eccentrically. W. K. Hatt used specimens of I-bar type having a  $4 \times 4$ -in. cross-section and a gauge length of  $17\frac{1}{2}$  in. He reported that reinforcement must be placed in the heads of such specimens to secure satisfactory results. W. H. Henby's tests were made on prisms 21 in. long and  $2\frac{7}{8} \times 3\frac{1}{2}$  in. in cross-sectional. A. P. Mills' specimens were  $6 \times 6 \times 18$ -in. prisms. These were gripped on all four 6-in. sides near the top and bottom by means of cast-steel envelopes formed like truncated pyramids. The envelopes were fastened to the test-pieces by means of hard maple wedges. A ball-and-socket connection was placed between the envelopes and the heads of the testing machine.

The effect of consistency on the tensile strength is well shown in Table 9.

**513. The Transverse Strength of Concrete.**—Table 11 is abstracted from the tests of W. B. Fuller \* on  $6 \times 6$ -in. beams loaded at the center over 60- and 30-in. spans. A silicious sand of which 100 per cent was less than 0.25 in., 50 per cent less than 0.028 in., and 0 per cent less than 0.003 in. in diameter was used. The crusher-run trap rock varied between 2.1 and 0.17 in. in size. All concrete was of dry consistency. The effect on the transverse strength of varying the proportion of

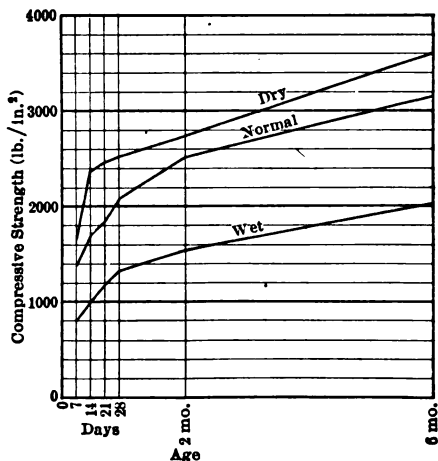


FIG. 10.—The Effect of Consistency on the Compressive Strength of 1:2:4 (by weight) Concrete Tested at Different Ages. Results were obtained at the Universities of Illinois, Wisconsin, California, and Massachusetts Institute of Technology. (Each point represents 15 or more tests on  $6 \times 6$ -inch cylinders.)

\* From Taylor and Thompson's *Concrete Plain and Reinforced*, 3rd Ed., p. 334.

cement is well brought out by the results in the right portion of the table. The results at the left show how the strength varies with the density, the percentage of cement being constant.

TABLE 11.—THE EFFECTS OF PROPORTION OF CEMENT AND DENSITY ON THE TRANSVERSE STRENGTH OF CONCRETE. (FULLER)

	PROPORTION OF CEMENT CONSTANT.						PROPORTION OF CEMENT VARIABLE.			
Mix by Wt. ....	1 : 1 : 5	1 : 2 : 4	1 : 3 : 3	1 : 4 : 2	1 : 1 : 3	1 : 2 : 3	1 : 2 : 5	1 : 2 : 6	1 : 3 : 6	1 : 4 : 7
Density . . . . .	0.858	0.865	0.829	0.760	0.822	0.844	0.809	0.865	0.848	0.847
Mod. of Rupt. (Lb./In. <sup>2</sup> ) . . .	504	439	355	210	655	471	380	319	226	181

The results of transverse tests by the U. S. G. S. on 8×11-in. concrete beams tested over a 12-ft. span are recorded in Table 8. These beams were loaded at the "third points" of the span. The experiments illustrate that the effect of consistency on the transverse strength is similar to its effect on the compressive strength. Portions of these beams retested over spans of about 6 ft. gave the same average modulus of rupture, all consistencies considered, at an age of one month.

Tests on 6×6-in. beams of 1 : 2 : 4 concrete which were made from the same batches as the tensile test-pieces of Table 10 were also reported by Withey. These specimens were loaded at the middle over a 3-ft. span. From 25 tests the average modulus of rupture was 352 lb. per square inch. Individual values ranged from 248 to 448 lb. per square inch. The ratio of the average compressive strength to the average modulus of rupture was 5.51 : 1 and the ratio of the average modulus of rupture to the average tensile strength was 1.86 : 1.

**514. The Shearing Strength of Concrete.**—Failure of concrete in pure shear is a rare occurrence, although the term "shear failure" is commonly used. When this term is applied to the diagonal failure in the web of a concrete beam it is misleading, for the cause of such failure is a tensile stress resulting from the combining of tensile and shearing stresses. On account of the difficulty of producing a pure shear stress but few tests have been made. The usual method of making such tests is to support the specimen as a cantilever or fixed beam and apply the load as closely as possible to the supports. Frequently encircling envelopes are placed at each support and around the span portion to prevent bending of the test-piece.

In E. Mörsch's *Eisenbetonbau* (translation by E. P. Goodrich), Ch. III, the theory is advanced that the shearing strength of concrete ( $S_s$ ) is the geometric mean of its tensile ( $S_t$ ) and compressive strength ( $S_c$ ),  $S_s = \sqrt{S_t \times S_c}$ . Mörsch's results obtained from tests on 7×7×16-in.

prisms loaded as indicated in Fig. 11 are in accordance with this theory. However, since in these tests the concrete cracked in tension before the full shearing strength was developed, one is led to believe that the computed values of shear are much too low. Mörsch also tested the torsional strength of solid concrete cylinders 10.24 in. in diameter by 13.38 in. long and hollow cylinders of the same outside measurements with an internal diameter of 5.9 in. The proportions were 1 : 4 gravel concrete and the ages of the solid and hollow specimens three months and two months, respectively. The modulus of rupture in torsion (see formula 16 in Art. 23) for four solid cylinders averaged 243 lb. per square inch, for three hollow cylinders 126 lb. per square inch. All specimens fractured on helicoidal surfaces. Tensile tests on like hollow sections gave a strength of 113.8 lb. per square inch. Apparently the torsional strength is equal to the tensile strength if the specimen is dimensioned so that the intensity of the shearing stress is approximately the same over the entire cross-section. Mörsch also reports results of tests on slotted beams reinforced in the tension fibers so that failure in horizontal shear along a small portion of the neutral plane was produced. Here again it seems certain that diagonal tension must have largely influenced the results.

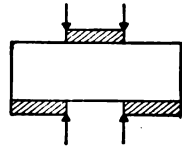


FIG. 11. — Mörsch's Shear Specimen.

The results of shearing tests on 5-in. cylinders of concrete made at the Mass. Inst. of Technology under the direction of C. M. Spofford\* are digested in Table 12. In these tests the end and center portions of the cylinders were tightly clamped so that bending action was made small. The results indicated such an abnormally high ratio of shearing to compressive strength that one is led to believe frictional resistance must have been induced at the planes of fracture. Specimens cured in water for one month appear to be slightly stronger in shear than those cured in air.

The most comprehensive series of tests done in this country are those of Talbot.† His experiments included punching tests on plane plates, on recessed plates, and on plates reinforced and recessed as shown in Table 12. The punch used was a cylinder  $5\frac{7}{8}$  in. in diameter which acted through the specimens against a die 6 in. in diameter. Talbot reported that the reinforced recessed plates were the most satisfactory of the types of specimen used in the punching tests. He also used short beam test-pieces restrained at the end in a device like the Johnson shear tool Art. 56 and applied the load across the span uniformly but without restraining the center portion of the beam.

From the meager information now in hand it appears that the true

\* See Reid's *Concrete and Reinforced Concrete Construction*, p. 198.

† *Bulletin No. 8, University of Illinois.*



TABLE 12.—SUMMARY OF RESULTS OF SHEAR TESTS ON BROKEN  
STONE CONCRETE MADE AT UNIVERSITY OF ILLINOIS AND MASSA-  
CHUSETTS INSTITUTE OF TECHNOLOGY

Mix by Vol.	Stored in.	No. Tests.	STRENGTH IN LB. PER SQ. IN IN		Ratio Shear* Compr.	Form of Specimen.
			Shear.	Compression.*		
UNIVERSITY OF ILLINOIS TESTS. AGE 2 MONTHS.						
1 : 3 : 6	Air	9	679	1230	0.55	
1 : 3 : 6	Water	7	729	1230	0.59	
1 : 3 : 6	Damp Sand	4	905	2428	0.37	
1 : 2 : 4	"	5	1193	3210	0.37	
1 : 3 : 6	Air	17	796	1230	0.65	
1 : 3 : 6	Water	5	879	1230	0.71	
1 : 3 : 6	Damp Sand	4	1141	2428	0.47	
1 : 2 : 4	"	5	1257	3210	0.39	
1 : 3 : 6	Air	4	1051	1230	0.86	
1 : 3 : 6	Damp Sand	4	1821	2428	0.75	
1 : 2 : 4	"	5	2145	3210	0.67	
1 : 3 : 6	Damp Sand	4	1313	2428	0.54	
1 : 2 : 4	"	6	1418	3210	0.44	4×4×12 in. beams restrained at ends
TESTS AT MASS. INST. OF TECH. AGE 1 MONTH.						
1 : 2 : 4	Air	.....	1310	2070	0.63	
1 : 2 : 4	Water	.....	1650	2620	0.63	
1 : 3 : 5	Air	.....	1240	1310	0.94	
1 : 3 : 5	Water	.....	1120	1360	0.82	
1 : 3 : 6	Air	.....	1180	950	1.25	
1 : 3 : 6	Water	.....	1120	1270	0.88	

\* Illinois specimens were 6-in. cubes; M.I.T. specimens were 5×18-in. cylinders.

NOTE.—In Illinois tests limestone passed 1-in. mesh. Trap rock passing 1½ in. ring was used at M. I. T.

shearing strength of concrete lies between 40 and 60 per cent of the compressive strength.

**515. The Effect of Fatigue on Concrete.**—Tests reported by J. L. Van Ornum in the *Trans. A. S. C. E.*, Vol. 58, p. 294, indicate that the ultimate compressive strength of concrete under a large number of rapidly repeated loadings is only one-half of its strength under a progressively applied static loading. He obtained a similar relation for reinforced concrete beams.

**516. The Strength of Cinder Concrete.**—On account of its fireproofing properties and its light weight, cinder concrete is considerably used for floor construction and for a fireproofing shell around beams and columns.

TABLE 13.—THE COMPRESSIVE STRENGTH AND SPECIFIC WEIGHT OF CINDER CONCRETE

No. Spec.	Mix by Vol.	Consistency	Age in Months.	Compressive Strength, Lb. per Sq. In.	Specific Weight, Lb. per Cu. Ft.	Type of Specimen.	How Cured.	Authority.
3	1 : 1 : 3	Dry	1	1466	112.1	12-in. cubes	Air	<i>Tests of Metals.</i> 1908
3	1 : 1 : 3	Dry	3	2001	110.4			
3	1 : 2 : 3	Dry	1	1098	115.2			
3	1 : 2 : 3	Dry	3	1634	112.8			
3	1 : 2 : 4	Dry	1	904	111.2			
3	1 : 2 : 4	Dry	3	1325	107.9			
3	1 : 2 : 5	Dry	1	769	108.8			
3	1 : 2 : 5	Dry	3	1084	105.3			
3	1 : 2 : 6	Dry	1	529	107.6			
3	1 : 2 : 6	Dry	3	788	103.5			
3	1 : 2 : 5	Wet	1	1081	116.5	8×16-in. cyl.	Moist air	U.S. Geol. Sur. Bull. No. 344.
3	1 : 2 : 5	Medium	1	1201				
3	1 : 2 : 5	Damp	1	1118				
3	1 : 2 : 5	Wet	3	1764				
3	1 : 2 : 5	Medium	3	1819				
3	1 : 2 : 5	Damp	3	1726				
3	1 : 2 : 5	Wet	6	2021				
3	1 : 2 : 5	Medium	6	2203				
3	1 : 2 : 5	Damp	6	1945				
10	1 : 2 : 5	Consistency not stated. Concrete taken from 4 buildings in N. Y. City.	1	407	107	8×16-in. cyl.	Air	Strehan and Perrine in <i>Engr. News</i> Vol. 70 p. 722
10	1 : 2 : 5		2	701				
10	1 : 2 : 5		6	933				
10	1 : 2 : 5		1	818				
10	1 : 2 : 5		2	1254				
10	1 : 2 : 5		6	1744				
10	1 : 2 : 5		1	980				
10	1 : 2 : 5		2	1035				
10	1 : 2 : 5		6	1478				
10	1 : 1 : 5*		1	507				
10	1 : 1 : 5		2	662	100			
10	1 : 1 : 5		6	754				

\* Hand-mixed, the other batches of this group were machine-mixed.

The strength of cinder concrete is decidedly variable and is greatly influenced by the strength, granulometric composition, and absorptive properties of the cinders. Table 13 shows the results of tests made by various experimenters. The results of the tests by the U. S. G. S. show that consistency has but little effect on the compressive or transverse strength of cinder concrete. From the latter tests on 6-in. cubes and 8×11-in. beams on a 12-ft. span, the ratio of the compressive strength to the transverse modulus of rupture averages 9.8 : 1 for concrete six months old.

TABLE 14.—THE COMPRESSIVE STRENGTHS OF SLAG CONCRETES OF 1 : 3 : 6 : PROPORTIONS BY VOLUME

Sand.	River	No. 1 Slag	No. 2 Slag	River	No. 1 Slag	No. 1 Slag	River	No. 1 Slag	No. 2 Slag
Coarse Aggregate	No. 1 Bank Slag	No. 1 Bank Slag	No. 1 Bank Slag	Ma- chine Slag	Ma- chine Slag	Ma- chine Slag	No. 2 Bank Slag	No. 2 Bank Slag	No. 2 Bank Slag
Age	COMPRESSIVE STRENGTH IN L.B. PER Sq.IN.								
28 da. ....	1033	863	963	670	561	708	1028	697	837
90 da. ....	1377	1100	1228	928	826	994	1307	1040	1076
6 mo. ....	1478	1222	1363	854	983	1057	1440	981	1225
1 yr. ....	1722	1131	1309	902	1038	1052	1328	666	1232
2 yr. ....	1778	959	1367	1200	836	960	1888	1030	1348
Wt. in lb./ft. <sup>3</sup> . . .	138	121	131	129	120	122	137	129	128

TABLE 15.—PROPERTIES OF AGGREGATES IN SLAG CONCRETES

Element.	CHEMICAL ANALYSES.					MECHANICAL ANALYSES.			
	Per Cent of Element in					Mesh.	Per Cent Passing.		
	River Sand.	Slag Sand No. 1	Slag Sand No. 2	Bank Slag.	Ma- chine Slag.		River Sand.	Slag Sand No. 1.	Slag Sand No. 2.
SiO <sub>2</sub> . . . . .	79.28	30.31	32.24	37.26	31.28	10	87.5	75.0	75.4
Al <sub>2</sub> O <sub>3</sub> . . . . .	8.23	20.72	16.32	16.62	17.40	20	74.6	36.3	34.4
CaO . . . . .	5.60	44.67	46.51	39.26	46.20	30	50.0	15.6	16.0
MgO . . . . .	0.92	1.48	1.45	3.06	1.43	40	28.1	9.8	11.3
Fe <sub>2</sub> O <sub>3</sub> . . . . .	0.59	1.43	1.91	2.50	1.52	50	17.2	7.8	10.2
SO <sub>3</sub> . . . . .	5.38	1.39	1.57	1.30	1.56	80	1.6	4.3	5.5
						100	0.8	3.1	3.5
OTHER PROPERTIES.									
Size . . . . .				No. 1 1"-1"	No. 2 1"-2 1/4"	1" - 2"			
Wt. in lb./ft. <sup>3</sup> .	106	55	49	67	72	96			

It will be noted that this ratio is considerably larger than the corresponding value for gravel or broken stone concrete.

**517. The Strength of Slag Concrete.**—For purposes of construction similar to those mentioned in discussing cinder concrete, slag concrete is sometimes used. The results given in Tables 14 and 15 are taken from *Furnace Slags* issued by the Carnegie Steel Co., Pittsburgh. Each compressive strength result is the average of three tests on 12×16-in. cylinders. All specimens were made of standard Portland cement. They were daily sprinkled with water for twenty-seven days after making. The results show that the slag sand No. 2 made by running the molten slag into a water vat is slightly superior to slag sand No. 1 which was run through a spray as it fell into the water vat. The crushed bank slags, No. 1 and No. 2, produced a stronger concrete than the machine slag which was disintegrated by sprinkling with water after it had partially cooled.\*

#### THE ELASTIC PROPERTIES OF MORTARS AND CONCRETES

**518. General Characteristics of the Elastic Curves.**—As Fig. 12 shows, the stress-deformation curves for cement and concrete resemble those for other brittle materials like cast iron, brick and stone. Carefully made experiments fail to disclose a limit of proportionality or an elastic limit. Compression tests indicate, however, that after several applications of stresses less than one-half to two-thirds of the ultimate strength of concrete the set becomes constant. On account of the occurrence of set for low intensities of stress the true elastic stress-deformation curve for these materials may differ considerably from the gross deformation curve. Bach, in finding the true elastic curve in compression repeatedly applied and released each load until the set at zero load became constant. By subtracting the set from the total deformation the elastic deformation for a given load was determined. Although the true elastic curve is of importance in considering the change of shape of concrete after removal of stress, it does not afford information on the amount of total deformation which the material undergoes for a given unit stress. Since in reinforced concrete construction the unit stress carried by the steel will depend upon the gross deformation of the combination, more application is made of values derived from the gross-deformation curve than of those gotten from elastic deformations.

**519. Calculation of the Modulus of Elasticity.**—Since mortar and concrete have no elastic limit, the modulus of elasticity must be the slope of the stress-deformation curve at zero stress. For mixes which have a stress-deformation curve of sharp curvature near the origin, the

\* For additional tests on slag concrete, see *Engr. News*, Vol. 72, p. 103 and Vol. 75, p. 276.

initial modulus of elasticity,  $E$ , is of little value, except for comparing the stiffness of different concretes; since for all finite values of unit stress,  $S$ ,

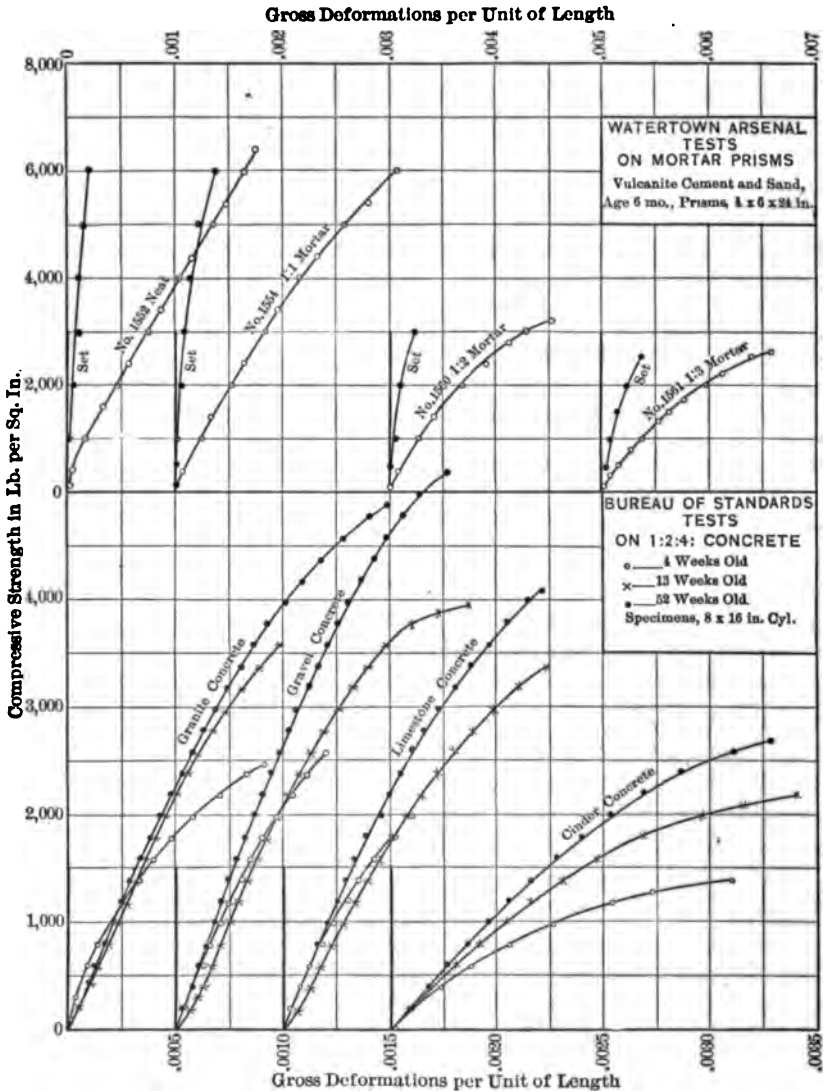


FIG. 12.—Stress-deformation Diagrams for Mortars and Concretes in Compression.  
(*Tests of Metals*, 1904. *Technologic Paper No. 2.*)

or unit deformation,  $\epsilon$ ,  $E\epsilon > S$  and  $\frac{S}{E} < \epsilon$ , as may be seen from Fig. 13. For such curves the slope of a secant drawn from the origin through a

point corresponding to a working unit stress is useful, because the simple relation proposed by Hooke can be applied for neighboring stresses. Also, in reinforced concrete design the use of the secant modulus considerably shortens the computations without undue sacrifice of accuracy.

Prof. Bach's studies of true elastic curves led to an exponential equation for unit deformation in compression,  $\epsilon = KS_c^m$ , in which  $K$  and  $m$  are constants depending upon the material. Mörsch \* gives the following equations of the true elastic curve for 1 : 2½ : 5 sand and gravel concrete and 1 : 2½ : 5 sand and broken-stone concrete,  $\epsilon = \frac{S_c^{1.14}}{5,676,100}$  and  $\epsilon = \frac{S_c^{1.16}}{9,190,500}$ , respectively.

Prof. Talbot † developed a formula for the elastic curve in compression, based upon the theory that the stress-gross-deformation curve for concrete is a parabola. His formula is  $S_c = (1 - \frac{1}{2}q)E_c\epsilon$  in which  $q$  is the ratio of  $\epsilon$  (the unit deformation for  $S_c$ ), to the ultimate unit deformation, and  $E_c$  is the initial modulus.

## 520. Values of the Modulus of Elasticity of Mortars and Concretes.—

Table 16 shows values of the moduli of elasticity of neat cement, mortars and concretes compiled from the sources mentioned. It will be noted that the values for neat cement are decidedly variable and that air-cured neat cement and rich mortars have a much lower modulus than similar mixes cured in water. The modulus increases with the density, and to some extent with age, if specimens are water-cured. Mörsch's results show that the moduli for wet mixes are less than for dry mixes of the same proportions. For lean mixes of the same aggregate, the modulus like the strength increases with the proportion of cement; but the variation is small for proportions richer than 1 : 2, in the case of mortar, or 1 : 1½ : 3, in the case of concrete. For rich mixes, however, the curvature of the stress-deformation line is very slight throughout the range of working unit stress (Fig. 12).

The stiffness of the coarse aggregate also considerably affects the rigidity of the concrete. Furthermore, a large difference in the moduli of elas-

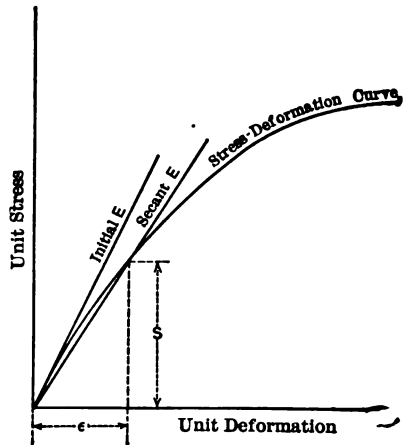


FIG. 13.—Method of Finding the (Secant) Modulus of Elasticity,  $E = \frac{S}{\epsilon}$ .

\* *Eisenbetonbau*, p. 22 (trans.).

† *Bull.* No. 14, Engr. Expt. Sta. Univ. of Ill.

TABLE 16.—MODULI OF ELASTICITY FOR NEAT CEMENT, MORTAR AND CONCRETE

No. Spec.	Mix by Volume.	Age in Mo.	Method of Curing.	Consistency.	Cement and Aggregate.	Dimensions of Spec., in In.	MODULUS OF ELASTICITY LB. PER SQ. IN.		Range of Stress, (Lb./In. <sup>2</sup> )	Authority.
							Tension.	Compression.		
1	Neat	1	Air	Plastic	Alpha	4×6×24		2,940,000	100 to 600	Tests at Watertown Arsenal.
1	"	6	"	"	Alpha	"		2,940,000		
1	"	2½	"	"	Atlas	"		1,610,000		
1	"	2	"	"	Lehigh	"		1,670,000		
1	"	2½	"	"	Peninsular	"		3,450,000		
1	"	1	Water	"	Alpha	"		6,250,000	100 to 500	Tests of Metals, 1902, 1904, 1907
1	"	2	"	"	Lehigh	"		5,260,000		
1	"	2½	"	"	Peninsular	"		3,570,000		
1	1:1	2½	Air	"	Peninsular Cement and Sand	"		3,120,000		
1	1:1	"	Water	"		"		3,330,000		
1	1:1	"	Air	"		"		2,220,000		
1	1:2	"	Water	"		"		3,570,000		
1	1:2	"	Air	"		"		1,540,000		
1	1:3	"	Water	"	Sand and Trap rock, 1½ to ¾ in.	"		2,860,000	100 to 600	Mörsch
1	1:3	"	Air	"		"		1,210,000		
1	1:4	"	Water	"		"		1,670,000		
1	1:1½:3	11	Air	"		12×12×96		5,000,000		
1	1:2:4	11	"	"		"		4,390,000		
1	1:3:6	4	"	"	"		2,580,000	0-522 C. 0-44 T. elastic def.	Hatt	
3	1:1.8:1.2	3	Dry Wet Dry Wet	Dry	Sand and Gravel <0.8 in.	7×7×29	3,380,000			3,650,000
3	1:1.8:1.2	3		Wet		"	2,950,000			3,160,000
3	1:2.4:1.6	3		Dry		"	3,410,000			3,200,000
3	1:2.4:1.6	3		Wet		"	3,140,000			2,630,000
37 C. 27 T.	1:2:5	1	In dry Sand	Medium	Sand and Limestone	8×12-in. cyl. in C.	3,800,000	3,350,000	E. from elastic def.	
	1:2:5	3		"		4×4×30 in. prisms in T.	5,460,000	4,610,000		
	1:2:5	1		"			4,320,000	4,130,000		
	1:2:5	3		"			4,510,000	4,800,000		

4 C., 19 T. 4 C., 14 T. 4 C., 3 T.	1 : 2 : 4 1 : 2 : 5 1 : 3 : 6	1 to 3 }	Air	Plastic	Sand and Limestone	$2\frac{1}{2} \times 3\frac{1}{2} \times 21$ in. in T. $2\frac{1}{2} \times 3\frac{1}{2} \times 11$ in. in C.	$\left\{ \begin{array}{l} 3,700,000 \\ 3,600,000 \\ 3,100,000 \end{array} \right\}$	Initial E	Henby
3	1 : 2 : 5	1	Moist Air	Wet Medium Damp	Sand and Cinders	8 × 16 in. cyl.	$\left\{ \begin{array}{l} 1,240,000 \\ 1,350,000 \\ 1,290,000 \end{array} \right\}$		
3	1 : 2 : 5	1							
3	1 : 2 : 5	1							
3	1 : 2 : 4	1		Wet Medium Damp	Sand and Granite	"	$\left\{ \begin{array}{l} 3,570,000 \\ 4,080,000 \\ 4,890,000 \end{array} \right\}$		
3	1 : 2 : 4	1							
3	1 : 2 : 4	1		Wet Medium Damp	Sand and Gravel	"	$\left\{ \begin{array}{l} 3,790,000 \\ 3,880,000 \\ 4,070,000 \end{array} \right\}$		
3	1 : 2 : 4	1							
3	1 : 2 : 4	1		Wet Medium Damp	Sand and Limestone	"	$\left\{ \begin{array}{l} 3,590,000 \\ 3,430,000 \\ 4,260,000 \end{array} \right\}$		
3	1 : 2 : 4	1							
21	1 : 2 : 4	1					$\left\{ \begin{array}{l} 1,610,000 \\ 1,820,000 \\ 2,120,000 \end{array} \right\}$		
21	1 : 2 : 4	3		Medium	Sand and Cinders	"	$\left\{ \begin{array}{l} 2,110,000 \\ 4,290,000 \\ 4,430,000 \end{array} \right\}$		
21	1 : 2 : 4	6					$\left\{ \begin{array}{l} 4,480,000 \\ 4,770,000 \end{array} \right\}$		
21	1 : 2 : 4	12							
21	1 : 2 : 4	1			Sand and Granite	"			
21	1 : 2 : 4	3							
21	1 : 2 : 4	6							
21	1 : 2 : 4	12							
21	1 : 2 : 4	1			Sand and Gravel	"	$\left\{ \begin{array}{l} 4,500,000 \\ 4,620,000 \\ 5,240,000 \end{array} \right\}$		
21	1 : 2 : 4	3							
21	1 : 2 : 4	6							
21	1 : 2 : 4	12					$\left\{ \begin{array}{l} 5,396,000 \\ 3,590,000 \\ 3,810,000 \end{array} \right\}$		
21	1 : 2 : 4	1			Sand and Limestone	"	$\left\{ \begin{array}{l} 4,310,000 \\ 4,550,000 \end{array} \right\}$		
21	1 : 2 : 4	3							
21	1 : 2 : 4	6							
21	1 : 2 : 4	12							



ticity for the mortar and aggregate must be accompanied by non-uniform distribution of stress when the concrete is loaded. Such non-homogeneity in elasticity undoubtedly accounts for the low early strengths exhibited by concretes made from certain classes of granite and trap rock.

From the results now available it appears that the tensile modulus of elasticity is slightly less than the compressive modulus for gravel concrete. For broken stone concrete the difference is negligible.

**521. Poisson's Ratio for Concrete.**—Under compression the unit lateral expansion of concrete is about one-sixth to one-twelfth of the expansion in the direction of the applied forces for the ordinary range of working stress. The ratio increases with the richness of the mix and is influenced by the other factors which affect the magnitude of the modulus of elasticity. Values of Poisson's ratio in compression found by Talbot \* varied between 0.1 and 0.16 for working loads on 1 : 2 : 4 concrete sixty days old. Withey † found, for stresses at one-fourth of the ultimate strength on concrete two months old, the following values: For 1 : 3 : 6 mix, 0.08; for 1 : 2 : 4 mix, 0.11; for 1 : 1½ : 3½ mix, 0.18; for 1 : 1½ mortar, 0.16.

**522. Expansion and Shrinkage Due to Variations in Moisture Content.**—The changes in volume of neat cements and mortars while hardening has already been considered in Art. 357. Only a few experiments on plain concrete have been reported, but from them one must conclude that concrete also increases in volume when hardened under water and shrinks in air. These changes are more pronounced with the rich mixes than with lean ones. The changes increase with the age of the specimen; but the rates of change in volume decrease, becoming very small after three months of storage under constant conditions. Wet mixtures hardening in air contract more slowly than dry mixtures. Alternate exposures to moisture and dry air cause alternate expansions and contractions in the concrete. Such changes take place regardless of the age of the concrete. It is probable, however, that the character of the aggregate from which the concrete is made to some extent influences the magnitude of these changes.

The results of experiments made by A. T. Goldbeck, of the Dept. of Public Roads, U. S. Dept. Agriculture,\* appear in Table 17. In these tests, columns 8×8×60 in. were cured either in air or under two thicknesses of burlap dampened twice a day for the periods indicated. Readings were taken on both sides of the specimens over a 50-in. gage length by means of micrometer-screw extensometers reading to 0.0001 in. The first measurements were made as soon as the concrete had set. The materials were Old Dominion Portland Cement, river sand of which 97

\* *Bull.* No. 20, Univ. of Illinois.

† *Bull.* No. 466, Univ. of Wisconsin.

and 34 per cent. passed  $\frac{1}{4}$  in. and No. 30 sieves, respectively, and crushed gneiss nearly all of which passed a 1-in. sieve and was retained on a  $\frac{1}{8}$ -in. mesh.

Tests on concrete made of Giant and Universal Portland cements, bar sand, and run-of-crusher trap rock passing a  $1\frac{1}{4}$ -in. mesh have been reported by H. C. Berry; \* they are also summarized in Table 17. Specimens  $9 \times 9$  in. with a 20-in. gage length were employed in these experiments. Readings were taken with a Berry strain-gage on four sides of each specimen, the initial observations being made within eighteen hours after placement of concrete.

TABLE 17.—THE CHANGES IN LENGTH OF CONCRETE PRISMS IN HARDENING .

Mix by Vol.	Consistency.	Method of Curing.	CHANGE PER UNIT LENGTH (* = EXPANSION) AT				Authority.
			10 Days.	30 Days.	60 Days.	150 Days.	
1 : 2 : 4	Very Dry	Air	.00014	.00034	.00048		Gold- beck.
1 : 2 : 4	Very Wet	"	.00011	.00030	.00041 <sup>1</sup>		
1 : 3 : 6	Very Dry	"	.00014	.00038	.00053		
1 : 3 : 6	Very Wet	"	.00010	.00028	.00042		
1 : 2 : 4	Medium Wet	Under	*.00011	.00010	.00025 <sup>1</sup>		
1 : 2 : 4	Very Wet	moistened	*.00004	.00000	.00016		
1 : 3 : 6	Very Dry	burlap for	*.00012	.00006	.00023 <sup>1</sup>		
1 : 3 : 6	Very Wet	15 days	*.00008	.00002	.00027		
1 : 2 : 4	Very Wet	Under burlap contin- uously	*.00005	*.00005	*.00002 <sup>1</sup>		
1 : 2 : 4	Wet	Air	.00005	.00016		.00023	Berry
1 : 2½ : 5	Wet	Air	.00002	.00009		.00017	
1 : 2 : 4	Wet	Water	.00001	*.00009		*.00020	
1 : 2½ : 5	Wet	Water	*.00003	*.00007		*.00002	

<sup>1</sup> Age of specimen = 50 days.

Observations on the Chevy Chase † experimental concrete road furnished the interesting information tabulated below. This road is 20 feet wide and 6 in. thick. The proportions of the concrete were  $1 : 1\frac{1}{2} : 3\frac{1}{2}$ . The oil-cement concrete contained 5 pints of oil per bag of cement. Coarse aggregates were screened through a  $1\frac{1}{4}$ -in. and retained on a  $\frac{1}{4}$ -in. mesh. The concrete was covered with a wet canvas for twenty-four hours and then covered with a 2-in. coat of sand which was sprinkled twice a day for

\* *Proc. A. S. T. M.*, Vol. 11, p. 563.

† Built by U. S. Dept. of Public Roads, see *Concrete Cement Age*, Vol. 4, p. 97.

eight days. This sand coat was left on for fifteen days, after which the road was opened to traffic. The road was built during the fall of 1912 and spring of 1913, and the observations recorded below were made about one year after completion.

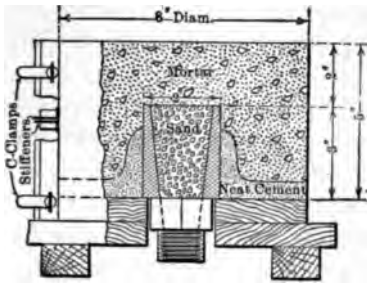
Concrete.	Aggregate.	Length, Ft.	Average Distance Between Cracks In Ft.
Plain.....	Gravel.....	425	61
Plain.....	Limestone.....	600	100
Oil-cement.....	Limestone.....	210	100
Oil-cement.....	Gravel.....	260	45
Oil-cement.....	Trap.....	290	60
Plain.....	Trap.....	300	40

#### THE PERMEABILITY AND ABSORPTION OF MORTAR AND CONCRETE

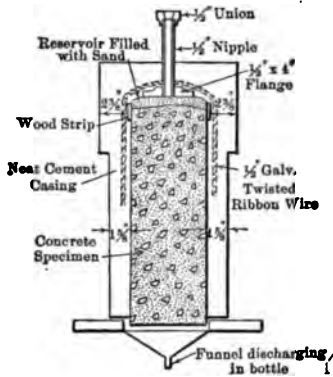
**523. Discussion of Terms.**—Water may enter a porous body through capillary attraction, it may be forced in under pressure, or it may be introduced by a combination of pressure and capillary attraction. The character and size of the minute canals connecting the pores with one another and with the exterior of the body are apparently the factors which determine the rate of flow into the body. By absorption is meant the drawing in of water by capillary attraction. All cement mixtures absorb water to some extent. When the size and arrangement of the canals are such as to permit water to flow through the substance it is said to be permeable. The rate of absorption is dependent upon the size, number of pores connected with the surface of the body, and upon the size and character of the connecting ducts; the permeability is dependent upon the character and size of the minute passageways leading through the body, consequently there can be no relation between these properties.

With hydrostatic heads of 100 ft., tests have been made which indicate that neither Portland cement paste nor mixtures made from it are absolutely impervious. Nevertheless, there is abundant evidence which shows that concrete and mortar can be made so impermeable that no leakage or dampness is visible on the surface opposite to the water pressure. Apparently, even when the humidity is high, the frictional resistance to flow prevents the water from leaving the free surface of such material at a rapid enough rate to escape evaporation. Concrete or mortar similar to the above will hereafter be frequently referred to as impermeable or impervious. The error in such use of these words should, however, be borne in mind.

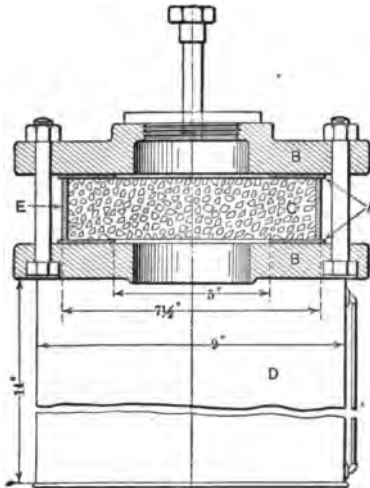
**524. Methods of Testing Permeability.**—Two methods of measuring permeability are in use. In one, the water passing out of the specimen is



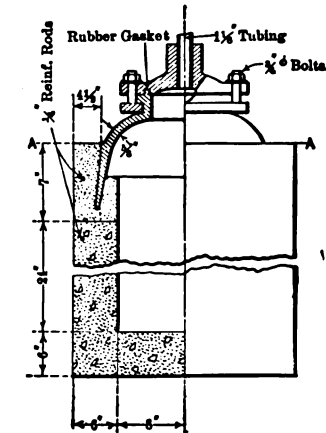
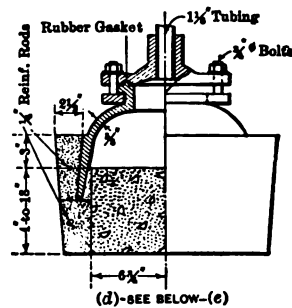
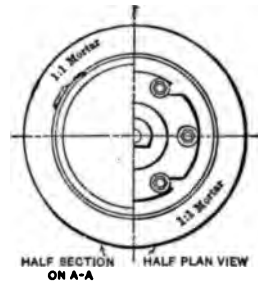
(a) Sectional Elevation of Mortar Permeability Specimen, not Suitable for Concrete. (University of Wisconsin.)



(b) Sectional Elevation of Specimen Used by Fuller and Thompson.



(c) Sectional Elevation of Bureau of Standards' Specimen. (AA = rubber washers; BB = cast-iron top and bottom of holder; C = specimen; D = retainer for leakage; E = wrought-iron pipe.)



(e) Used at Wisconsin to Measure Flow Perpendicular to Direction in which Concrete was Poured. Type (d) was used to measure flow parallel to direction of pouring.

FIG. 14.—TYPES OF PERMEABILITY SPECIMENS.

measured; in the other, the quantity of water entering the specimen is determined. The first method does not require as elaborate apparatus as the second, but is inaccurate when the leakage is small on account of evaporation losses. The second method necessitates the use of a calibrated water gage for measuring the flow and air pressure to force the water into the specimens. Measurements made by the latter method may include both the water absorbed and the water passing through the specimen, depending on the method of curing the test-pieces.

Fig. 14 shows five forms of specimen which, more or less modified, have been used in many important experiments. The type illustrated in Fig. 14a is the simplest, most easily made and readily attached to the testing apparatus; but the path of flow is neither restricted to a definite volume of concrete, nor is the area of the opening in the casting sufficient for the testing of concrete made from large aggregate.

In Fig. 14b is shown the form used by Fuller and Thompson in their experiments at Jerome Park. The use of this form of test-piece requires that the concrete or mortar be molded before the neat cement shell, since no provision is made for cleaning the surface of the concrete exposed to pressure. The use of this type of test-piece also necessitates that the core be thoroughly soaked and have its surface roughened before encasement in order that flow between lining and shell may be avoided.

Fig. 14c illustrates the form of mortar test-piece used by the Bureau of Standards. Similar specimens 18 in. in diameter were used in testing concrete. This type of test-piece requires expensive castings and gaskets. If the flow into the specimen is to be measured it is probable that difficulty in securing a tight joint at the gasketed surfaces will be experienced when high pressures are used. Furthermore the area of the openings for the entrance and exit of the water are only about one-fourth of the cross-sectional area of the specimen. On the other hand, no mortar shell is required with this test-piece and top and bottom surfaces of the specimen may be easily cleaned before placement on the apparatus.

A type of specimen successfully used at the University of Wisconsin appears in Fig. 14d. The apparatus is rather costly, but the labor of making and testing the specimens is not great. In molding, the mortar shell and concrete core are formed simultaneously. This type of test-piece permits cleaning or treatment of the exposed surfaces and the casting provides a reservoir for curing water.

Fig. 14e shows a modified form of specimen used at the above university for determining the flow in directions perpendicular to the pouring.

In permeability testing great care must be exercised to remove laitance and the excess mortar which flushes to the surfaces of the specimen. Precaution should also be taken to avoid leakage of water in molding the test-pieces. Great pains must be taken to procure uniformity and homo-

geneity in specimens equipped with mortar shells. If water gages are used and the flow into the specimens is measured the apparatus must be absolutely tight. Allowance must be made for changes in the height of the water columns due to temperature and absorption of water by the compressed air or vice-versa. Data for these corrections can be gotten by taking readings on a water-tight blank specimen holding the same amount of water as those under test and subjected to identical conditions.

**525. The Effect of the Proportion of Cement on Permeability.**—Data from tests show that the flow through concrete or mortar decreases with the increase in proportion of cement provided the density is constant. Plastic neat cement and 1 : 1 mortar linings 2 in. or over in thickness may be considered impervious under heads less than 100 ft., provided the mortar is not cracked by overstressing. If made of bank sands of good quality, 1 : 2 mortars of plastic consistency will, in general, show no leakage under a like pressure. With well-graded sands impervious mortars of 1 : 3 and 1 : 4 proportions may be secured. Fig. 15 \* well shows the effect of the proportion of cement upon the permeability of mortars made of fine (*Sd 4*), medium (*Sd 8*), and coarse (*Sd 9*) sands. The experiments were made on specimens like Fig. 14a. The properties of these sands may be found in Table 4, Ch. XIII.

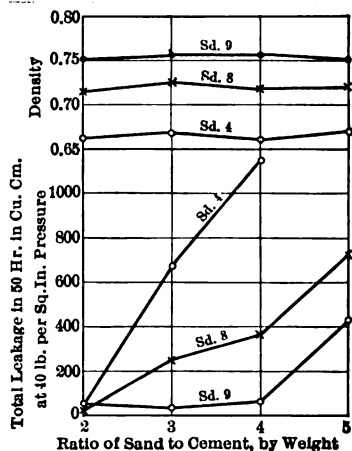


FIG. 15.—The Influence of Proportion of Cement on the Permeability of Mortar Made from Fine (*Sd*<sub>4</sub>), Medium (*Sd*<sub>8</sub>), and Coarse (*Sd*<sub>9</sub>), Sands.

A report from the University of Wisconsin on tests of gravel concrete varying in richness of mixture may be found in *Trans. Western Soc. Engr.*, Vol. 19, p. 833. It shows that if properly graded aggregate is used and proper precautions taken in mixing, placing and curing, concrete containing 20 per cent, of cement may be made water-tight for heads under 100 ft. A thickness of one foot should be sufficient to insure against leakage. For heads of 10 ft. and less a properly made 1 : 9 mixture 6 in. thick should be water tight.

**526. Effect of Density on Permeability.**—With the same proportion of cement, the permeability of concrete or mortar decreases as the density increases. The effects of variations in density are more marked with lean mixes than with rich ones. Therefore more attention must be paid to the grading of the aggregate for lean mixes than for rich ones.

\* *Proc. A. S. T. M.*, Vol. 13, p. 834.

From the results of numerous tests on sand mortars R. Feret concluded that most impermeable mortar for any given proportion of cement will be obtained when the percentage of fine sand (*F*) plus cement equals the percentage of coarse sand (*G*). This grading, he also found, produced the maximum strength and density (see Art. 502). Mortars made from fine sand will be found more impervious than those containing large grains (*G*) only.

Fig. 16 shows the relation between leakage and the ratio

$$\frac{\% \text{ cement}}{100(1 - \text{density})} \left[ = \frac{\% \text{ cement}}{100 (\text{air} + \text{water voids})} \right]$$

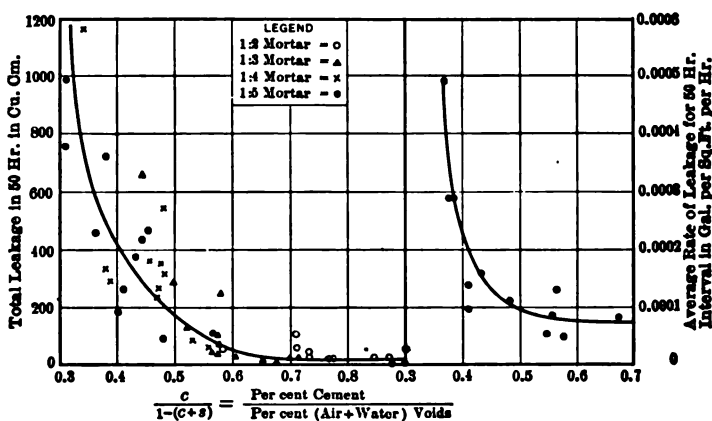


FIG. 16.—Relation of Leakage to Ratio  $\frac{\text{Vol. of Cement in Mix}}{\text{Vol. of Voids in Mix}}$  for Mortars (Left) and Concretes (Right) Made of Wisconsin Aggregates. Pressure was 40 lb. per square inch.

for Wisconsin tests on both mortars and concretes referred to in preceding article. From the shapes of the curves in the above figure and in Fig. 9, it is evident that the density of the mixture affects the permeability more than the strength. The methods of proportioning and grading the aggregate outlined in Art. 483 are of considerable value in securing water-tight concrete.

**527. The Effect of Consistency on Permeability.**—For mortars or concretes, the use of a too wet or too dry mix results in a decrease in imperviousness. This effect is most pronounced for lean mixes and is due principally to the decrease in density which results from the use of too much or too little water. An excess of water is preferable to a deficiency, but a plastic or mushy mixture, which can be compacted with little ramming and which shows moisture but no excess water when surfaced, is the most impervious.

The use of a dry consistency in conjunction with insufficient tamping forms the chief cause of permeability in concrete blocks. Tests at the University of Wisconsin have shown that a driving rain which will soak through a one-piece block made from a dry mix in a half-hour, will, in most cases, not pass through a similar block made from a plastic mix.

The relations between permeability, density and strength for : 1 : 2 $\frac{1}{4}$  : 4 $\frac{1}{2}$  broken stone concrete were well established by the experiment of Fuller and Thompson described in *Proc. A. S. T. M.*, Vol. 6, p. 358. Their tests show that the concrete of medium consistency had the greatest density and strength and was the least permeable.

The previously mentioned tests at the University of Wisconsin on gravel concrete show that it is better to make concrete too wet rather than too dry when imperviousness is desired. Owing to difficulty in tamping, dry mixtures of 1 : 1 $\frac{1}{2}$  : 3 proportions leaked badly at very low pressures, whereas medium and soupy mixtures were practically water-tight.

These tests also demonstrate that a soft, mushy consistency, which, when molded into a 6×12-in. cylinder, will slump about  $\frac{1}{2}$  in. on removal of the mold, is the most satisfactory. If such consistency is used and due attention is given to other conditions of fabrication, it is possible to make concrete of 1 : 9 proportions water-tight under 100-ft. head.

**528. Effect of Time of Mixing on Permeability.**—Obviously, the homogeneity of the mix is a most important factor in determining the imperviousness. Other tests at the University of Wisconsin (see previous citation) have shown that the time of machine mixing greatly influences the uniformity and magnitude of the flow. In these tests the different periods of mixing in a No. 0 Smith machine were three-quarters, two, and five minutes, from one-quarter to one-half minute of these intervals being devoted to dry mixing. The mixer was run at 30 r.p.m. The results indicate that a two-minute period was sufficient to secure good results with either wet or dry sand. Mixes of 1 : 3 : 6 proportions, by weight, which were impermeable when mixed two minutes, leaked badly when mixed only three-quarter minute. The leakage was increased by the use of damp sand. Placing the water in the mixer before admission of the dry materials caused great variation in results of individual tests and produced a much more permeable concrete than that gotten when a short period of dry mixing was allowed. The imperviousness of the richer 1 : 1 $\frac{1}{2}$  : 3 mixes, however, was not materially affected by the length of the mixing interval or by the omission of the dry-mixing period.

**529. Effect of Curing on Permeability.**—Proper curing is of the utmost importance if impermeable concrete or mortar is to be secured. Premature drying adversely affects the imperviousness of lean mixes much more than that of the rich mixes. The imperviousness of lean mixtures properly cured for three weeks may be entirely destroyed by drying



TABLE 18.—EFFECT OF CURING CONDITIONS ON THE PERMEABILITY OF CONCRETE

Materials: Universal Portland cement, Janesville sand; Janesville gravel (see Table 4).

All concrete was machine mixed.

All specimens were 6 in. thick.

After removal from the molds the surfaces of all air-cured specimens were well chipped and the test-pieces placed in a hall without being filled with water. Sprinkling was done with a hose each morning and evening as indicated. Artificial drying was accomplished by placing the specimens in an oven and raising the temperature in 2 or 3 hours to about 170° F. In drying specimens of batch 71 the temperature once rose to 250° F. for a period of about one hour.

Water pressure was 42 lb. per sq. in. (approx.)

No Spec.	Batch No.	Mix By Weight.	Per Cent Water Nominal.	Com- pressive Strength Lb. In. <sup>2</sup>	DAYS CURED IN				Average Rate of Leakage in Gallons per Sq. Ft. per Hr. for the Interval in Hr.	
					Molds.	Sprinkled.	Air.	Oven	0-50	40-50
4	57	1 : 3 : 6g	8.1	1505	2	26			.000286	.000210
4	69	1 : 3 : 6g	8.2	1150 <sup>a</sup>	1		27		.031400	.028500
4	70	1 : 3 : 6g	8.2	1595	1		27 <sup>b</sup>		.000516	.000330
2 <sup>c</sup>	87	1 : 3 : 6g	8.2	1510	2	28			.000440	.000300
2 <sup>c</sup>	87	1 : 3 : 6g	8.2	122.0	2	5	23		.007120	.005800
2 <sup>c</sup>	88	1 : 3 : 6g	8.2	1260	7 <sup>d</sup>		23		.012200	.012400
2 <sup>c</sup>	88	1 : 3 : 6g	8.2	1170	2	22			.024600	.024500
4 <sup>e</sup>	71	1 : 3 : 6g	8.2	1515	2	23			.020600 <sup>f</sup>	
1 <sup>g</sup>	71	1 : 3 : 6g	8.2	1515	2	23			.013600	.010700
2	95	1 : 3 : 2g : 5.8g	7.1	1550	2	26			.000218	.000090
2		1 : 3 : 2g : 5.8g	7.1	1550	2	26			.000356	.000180
2		1 : 3 : 2g : 5.8g	7.1				33 <sup>h</sup>		.000286	.000070
2		1 : 3 : 2g : 5.8g	7.1				33		.000420	.000160
2	86	1 : 3 : 2g : 5.8g	8.2	1120	2	27			.001510	.001330
2		1 : 3 : 2g : 5.8g	8.2				20 <sup>i</sup>		.001310	.000920
2		1 : 3 : 2g : 5.8g	8.2				33		.001260	.000800
2	104	1 : 3 : 2g : 5.8g	7.1		6		57 <sup>j</sup>		.004140	.003100
2	104	1 : 3 : 2g : 5.8g	7.1		6		57		.009500	.006800
		By Volume.								
2	110	1 : 2 : 4g	7.9	1530	1		28		.005440	.003660
2	110	1 : 2 : 4g	7.9	1935	2	27			.000120	.000050
2	111	1 : 2 : 4g	7.9	1765	4		25		.001610	.000870
2	111	1 : 2 : 4g	7.9	2040	6		23		.001100	.000600
2	113	1 : 2 : 4g	7.9	2570	13		15		.000373	.000165
2	113	1 : 2 : 4g	7.9	2620	20		8		.000347	.000140
2	114	1 : 2 : 4g	7.9	1750	2	3	24		.001360	.000775
2	112	1 : 2 : 4g	7.9	2906	2	56		3	.000594	.000180
4	77	1 : 1 : 3g	8.0	2935	2	26			.000080	.000035
4	80	1 : 1 : 3g	8.0	3262	1		26		.001900	.000890
2 <sup>k</sup>	89	1 : 1 : 3g	8.0	3460	2	26			.000021	.000014
1 <sup>l</sup>	89	1 : 1 : 3g	8.0	3700	2	5	21		.000140	.000088
2 <sup>c</sup>	103	1 : 1 : 3g	7.9	2810	6		22 <sup>l</sup>		.000150	.000097
1 <sup>l</sup>	103	1 : 1 : 3g	7.9	2990	6		22		.000326	.000200
2 <sup>c</sup>	102	1 : 1 : 3g	7.9	2850	6		57 <sup>m</sup>		.000218	.000130
2 <sup>c</sup>	102	1 : 1 : 3g	7.9	3550	6		57		.000682	.000250
1 <sup>l</sup>	102	1 : 1 : 3g	7.9				56		.000480	.000210
2	91	1 : 1 : 3g	7.9	3085	2	26			.000072	.000039
2	91	1 : 1 : 3g	7.9	3325	2	20			.000167	.000103

<sup>1</sup> These specimens were mixed  $\frac{1}{2}$  min. dry and  $4\frac{1}{2}$  min. wet; all others were mixed  $\frac{1}{2}$  min. dry and 1 $\frac{1}{2}$  min. wet.

<sup>2</sup> While being cured in air these specimens were filled with water.

<sup>3</sup> Results are for intervals from 0-44 and 20-44 hr. respectively.

<sup>4</sup> The tops of these specimens were sprinkled while in the molds.

<sup>5</sup> These specimens were filled with water 7 days before testing after having stood in air for 16 days. They were not sprinkled at any time however.

<sup>6</sup> These specimens were filled with water 10 days before testing.

<sup>7</sup> These specimens were filled with water after the 28-day test and stood in air till the 60-day test

<sup>8</sup> These specimens were full of water during the last 12 days of this period.

<sup>9</sup> Cylinders were removed to the hall after 4 days in the molds.

<sup>10</sup> L = One or more of these specimens leaked.

<sup>11</sup> M = One or more of these specimens were wet on bottom.

<sup>12</sup> D = One or more of these specimens were discolored by moisture on bottom.

at temperature of 120° to 150° F. Therefore structures which are to be subjected to alternate wet and dry periods should be made sufficiently thick to prevent drying, or the work should be sprinkled during dry periods. A short period of saturation following a period of dryness appears to decrease the permeability. The above statements are corroborated by evidence furnished from tests at the University of Wisconsin which are abstracted in Table 18 from the report previously cited. The form of test-piece used in these experiments is shown in Fig. 14d. Unless mentioned there was no visible evidence of flow on the surfaces of the specimens.

**530. Other Conditions Affecting Permeability.**—The water-tightness of concrete or mortar will also vary with the age of the concrete, intensity of pressure, thickness, character of the aggregate, condition of the surface to which pressure is applied, the quality of the water, and the direction in which the pressure is applied.

Test evidence shows that the rate of flow for mortars or concretes, which give visible signs of leakage, decreases very rapidly during the first month and somewhat less rapidly for the next two or three months. For specimens which are practically water-tight the decrease in flow between fourteen and twenty-eight days may be 50 to 80 per cent, but the change in flow after one month is small.\*

Tests on concrete by Taylor and Thompson show that the leakage varies directly with the intensity of the pressure for pressures between 20 and 80 lb. per square inch. Other experiments on concrete, by J. L. Davies, covering the same range of pressures, are confirmatory (see *Engr. News*, Nov. 7, 1912). A large number of tests by Hyde and Smith (*Jour. Franklin Institute*), Vol. 128, Sept., 1889 on neat, 1 : 1 and 1 : 2 mortars show, in general, that the rate of flow increases directly with the pressure for the range between 75 and 200 lb. per square inch.

With specimens which showed evidence of leakage, Taylor and Thompson found the rate of flow through concrete decreased as the thickness increased. For concrete specimens which show no visible leakage the rate of flow into the specimen does not appear to depend upon the thickness. The tests of the U. S. Bureau of Standards show little variation in permeability of mortar specimens varying from 1 to 3 in. in thickness.

The rate of flow is materially lessened by the presence of laitance or rich mortar on the surface exposed to pressure.

With water containing sediment or a high bacteria content the rate of flow diminishes rapidly with time. There also seems to be a gradual lessening of flow due to the plugging of the pores by efflorescence. This

\* The information following was compiled principally from *Technologic Paper No. 3*, U. S. Bureau of Standards, Wisconsin tests, and Taylor and Thompson in *Trans. A. S. C. E.* Vol. 59, p. 127.

is especially noticeable in dry-cured concrete, which leaks badly. Apparently calcium and magnesium hydroxide are evolved from the cement and carbonated when washed to the surface, thus forming a more or less impervious crust.

Concrete is somewhat more permeable to water when the pressure is applied perpendicular to the direction in which it is cast than when applied in the same direction. This is more pronounced for very wet mixtures placed in deep layers, since, under such conditions, non-homogeneity results from the settlement of the different constituents of the mix.

**531. The Absorption of Concrete and Mortar.**—A proper method of testing the absorptive properties of concrete or mortar is yet to be determined. By drying these substances at temperatures above 120° to 150° the imperviousness is greatly reduced, and consequently water will penetrate farther into such material than it would into undried concrete. Nevertheless, the method of conducting the absorption test ordinarily employed for other porous materials is generally used in testing mortar and concrete.

A series of tests on the absorption of 1 : 2, 1 : 4, 1 : 6 and 1 : 8 mortars made from three different sands is reported in *Technologic Paper No. 2*. The test-pieces were 2-in. cubes. They were stored in a damp room between testing periods. Before testing they were dried for several days at a temperature of 212 to 230° F. After cooling they were immersed in water to a depth of 3 in. They were periodically weighed until they gained less than  $\frac{1}{2}\%$  of 1 per cent. per day; the test was then stopped.

The experiments show that the absorption generally decreases with age, the greatest change occurring in the first two months. Mortars of damp consistency absorbed more than those of quaking consistency. Mortars made of coarse sand were somewhat less absorptive than those made of fine sands. The absorption, in general, decreased with the increase in richness of mix. The results show, however, that some factor, probably porosity, exercised a more important influence than the proportion of cement. Slag sand mortars, on account of the porosity of the aggregate, absorbed much more water than either the fine or coarse sand mortars.

Absorption tests made at the University of Wisconsin on mortars varying in richness from 1 : 2 to 1 : 5 have shown that those made from screenings and fine sands generally absorb more water than those made of better graded material. Mortars of 1 : 2 to 1 : 4 proportions made of good materials and properly cured ought not absorb more than 10 per cent of water after forty-eight hours' immersion. Well-made concrete of dense aggregate should not absorb more than 6 per cent of water under the same conditions.

**532. Waterproofing Materials.**—For a number of years attempts have been made to discover washes and compounds which will waterproof

concrete and mortar. In general, tests seem to show that if proper attention is given to a proportioning, mixing, and curing these compounds are not needed to secure impermeable concrete for heads under 100 ft. Furthermore, if good materials are procurable at average prices it is questionable if the extra expense involved in the use of such compounds will not be greater than the cost of additional cement required for water-tightness. It must also be understood that any beneficial results accruing from the use of these compounds cannot offset poor workmanship or improper curing. Furthermore, if the concrete cracks neither a properly made mix nor the use of such washes or compounds are effective. When there is probability of cracking, expansion joints should be used or a water-proof elastic membrane forming an integral part of the wall should be employed.

Waterproofing compounds may be divided into two classes: integral mixtures, those which are added before the concrete is mixed; and surface washes, those which are applied after the work is finished. The integral mixtures may be inert, simply void fillers such as finely ground clay or hydrated lime; or they may be active by virtue of compounds which they form during the hardening of the cement or by their repelling action toward water, petroleum residuum oil and the soap and alum combination, for examples. The coatings comprise paints and varnishes with a linseed-oil vehicle; bituminous compounds—the tars and asphalts; the hydrocarbons—the paraffin gasoline combination for example; and the soap solutions like alum and soap.

In *Technologic Paper No. 2*, U. S. Bureau of Standards, p. 23, the following comment regarding these classes of compounds appears: "As no organic substances can be considered as truly permanent the durability of all of these compounds, except the inert fillers, can be questioned." However, since several of these compounds have been more or less advocated and used in construction a very brief consideration of the results of tests on mixes containing some of them will be given.

**533. Effect of Hydrated Lime on Permeability.**—On account of the plasticity and easy working qualities which hydrated lime imparts to cement mortars, and since it also decreases segregation, it has been considerably used as a waterproofing compound. It may be mixed with the cement in proportions less than 15 per cent without producing loss in strength of concrete. Numerous tests on mortars and concretes have indicated that its use decreased permeability.\* With a 60-lb. per square inch water pressure, S. E. Thompson's tests \* indicated that additions of 8, 12 and 16 per cent of hydrated lime in terms of the weight of the cement gave water-tight concrete for 1 : 2 : 4, 1 : 2½ : 4, 1 : 3 : 5 proportions, respectively. The concrete was made of run-of-crusher hard conglomerate

\* *Proc. A. S. T. M.*, Vol. 8, p. 500.

rock below 2 in. size with dust remove. All of the sand passed a  $\frac{1}{2}$ -in. mesh, 26 per cent passed a No. 40 sieve and 4 per cent passed No. 100 sieve. Specimens of the type shown in Fig. 14a were used, the thickness of wall subjected to flow was 8 in., and leakage through the specimens was caught and weighed. Thompson contended that hydrated lime paste is a more efficient void filler than Portland cement paste, since the volume of the former is about  $2\frac{1}{4}$  times that of the latter.

Tests of lean concrete made by J. L. Davies \* on specimens 8-in. thick, similar to Fig. 14b, showed that the rate of flow was decreased by replacing 20 per cent of the cement, by weight, with high calcium lime. The results with dolomitic lime were not so satisfactory. Davies used mixtures varying from 1.1 : 3 : 6 to 1.3 : 3 : 6 by weight and pressures of 40 and 80 lb. per square inch. From a study of his results and the cost per cubic yard of the different concretes based on New York City prices, it does not appear that such use of lime is economical.

Tests by E. W. Lazell † and by the U. S. Bureau of Standards ‡ show that the replacement of 10 per cent of the cement with hydrated lime increases the imperviousness of 1 : 3 and 1 : 4 mortars.

**534. Effect of Finely Ground Clay on Permeability of Mortars.**—R. H. Gaines § made a number of tests on 1 : 3 mortars in which he replaced 10 per cent of Cowe Bay sand with finely ground clay. Under pressures of 80 lb. per square inch he found no leakage for test-pieces containing the clay, although the normal specimens leaked considerably. Davies also made tests on the efficiency of finely ground clay for waterproofing concrete, but he concluded that this method also was not economical.

**535. Integral Mixtures of Alum and Soap.**—In experiments at the University of Illinois || on 1 : 6 mortar under 40 to 45 lb. per square inch pressure the permeability was greatly reduced by the use of a soap and alum mixture amounting to 1.2 per cent of the cement. The experimenters recommended a 1 : 3 mixture of alum sulphate and hard soap. Compressive tests showed a small reduction in the strength of the mix due to the alum content. The lasting qualities of such treatment are, however, to be questioned.

**536. Oil Mixed Concrete.**—Recently there has been considerable agitation concerning the use of oil-mixed concrete. L. W. Page, of the Dept. of Public Roads, is the proponent of this material. He advocates ¶ the use of a petroleum residuum oil for which he draws the set of specifica-

\* *Engr. News*, Vol. 68, p. 866.

† *Proc. A. S. T. M.*, Vol. 6, p. 341.

‡ *Technologic Paper* No. 3, by R. J. Wig and P. H. Bates.

§ *Eng. News*, Sept. 26, 1907.

|| *Engr. News*, Vol. 62, p. 390.

¶ *Bulletin* No. 46, Office of Public Roads, U. S. Dept. of Agriculture.

tions below \*. He recommends 5 per cent of oil in terms of the weight of cement for dampproofing purposes, or 10 per cent of oil for waterproofing. His tests with 10 per cent of oil show that the time of setting is considerably lengthened, that the early strength is lowered, that the toughness and stiffness are not materially affected, but that the absorption is rendered very small and the permeability under low pressures is nil. Results of extensive tests by Feret † and by Taylor and Sanborn ‡ fail to substantiate the claims of Page concerning the waterproofing properties of oil-mixed concrete. The discrepancies, however, may have been due to differences in the character of the oils employed by the experimenters.

**537. Waterproofing by Surface Washes.**—The Sylvester surface washes have been considerably used, although the durability of the process is questionable. Before applying the washes, the surface of the concrete which is to be exposed to water pressure must be clean and dry. A boiling-hot aqueous solution of castile soap (9 per cent) is brushed over the surface, care being taken to avoid froth. After twenty-four hours a  $1\frac{1}{2}$  per cent aqueous solution of pure alum is applied cold. After another twenty-four hour period, the alternations in washes are repeated. The claim is made that the two solutions combine to form insoluble compounds which fill the voids in the surface. By applying several sets of washes reports § state that good results have been secured.

A neat cement or 1 : 1 mortar grout applied with a whitewash brush to the surface of the work exposed to pressure makes an impervious surface wash. To secure a good job the surface should be roughened and thoroughly soaked before the grout is applied. Three or four coats should be put on, care being taken to allow each coat to harden before applying the next. If sand is used it should be screened through a No. 20 sieve. For the best results the work should be frequently sprinkled after the grouting has set.

**538. Waterproof Membranes.**—Absolute imperviousness can be secured by the use of several layers of fabric like the better grades of roofing felt cemented to the work by hot asphalt or coal tar washes. To prevent

\* The oil shall have: 1, specific gravity 0.93 to 0.94 at 25° C.; 2, 99.9 per cent solubility in carbon disulphide at air temperature; 3, bitumen insoluble in 86° B. paraffin naphtha between 1.5 and 2.5 per cent; 4, residual coke between 2.5 and 4 per cent; 5, viscosity between 40 and 45 when tests are made on the first 100 c.c. emerging from an Engler viscosimeter after heating for 3 min. at 50° C.; 6, less than 2 per cent loss in weight when 20 grams is heated for five hours in cylindrical tin pan  $2\frac{1}{4}$  ins. deep and 1 in. high at a constant temperature of 163° C.

† *Engr. News*, Vol. 70, p. 1228.

‡ *Proc. A.S.C.E.*, Mar., 1913.

§ *Engr. Record*, Vol. 55, p. 395.

deterioration it is well to cover the alternate layers of pitch and felt with a protective layer of concrete or mortar.\*

#### THE EFFECTS OF TEMPERATURE ON MORTAR AND CONCRETE

**539. The Effect of Low Temperatures on Setting Concrete.**—Tests show that the rate of setting and hardening of concrete decreases as the temperature falls below 70° F. The rate of variation and way in which the set varies with the decrease in temperature differs with the cement used. Fig. 17 † shows strength-age curves for 1 : 2 : 4 gravel concrete

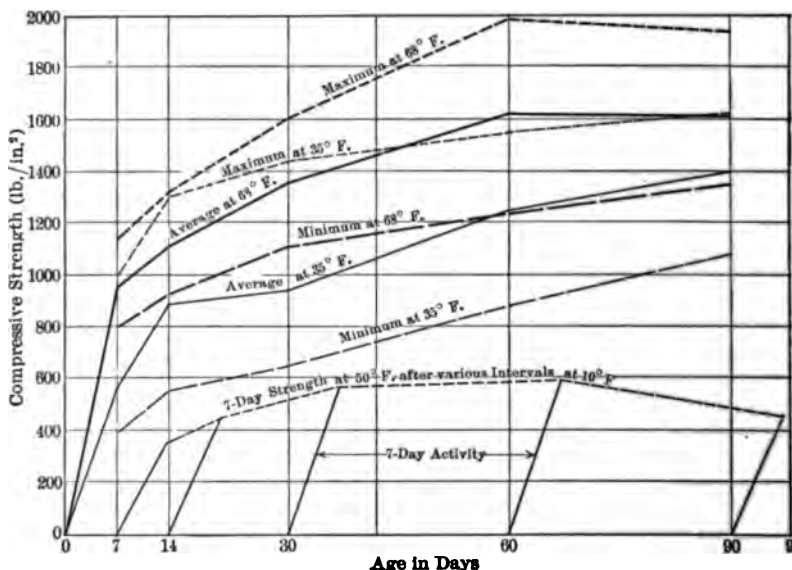


FIG. 17.—Effects of Low Temperatures on Compressive Strengths of 8-inch Cubes of 1 : 2 : 4 Gravel Concrete. Ten Brands of Portland Cement are Represented.

made from ten different brands of cement and cured at different temperatures. It will be observed that the maximum and minimum curves of strength for concrete cured at 68° F. are closer to the average than those for a temperature of 35° F. This well shows the range in the activity of different brands of cement at temperatures just above freezing.

If concrete is allowed to freeze before setting has begun, it will lie dormant until thawed. The long-time strengths of some Portland cements appear to be harmed by such procedure, others are affected but little.

\* For further information concerning methods of waterproofing concrete see *Proc. Nat. Asso. Cement Users*, Vol. 3, Vol. 5, p. 143, and Vol. 7, p. 667.

Results of tests on concrete waterproofed by fabrics may be found in *Bulletin No. 336* of The University of Wisconsin.

† Compiled from tests by the Universal Portland Cement Co. See *Engr. and Contracting*, Vol. 34, p. 448; also *Engr. Record*, Vol. 67, p. 66.

Natural cement concretes are badly injured by freezing prior to hardening. The results in the lower part of Fig. 17 show that freezing for different periods of time before initial set followed by a seven-day curing period at 50 to 60° F. produces a weaker concrete than would normally be obtained at an age of seven days. The magnitude of this weakening effect appears to be less when the freezing period is from one to two months than for either a shorter or longer time. Drying of the specimens may be responsible for the dropping of the curve between sixty and ninety days.

TABLE 19.—A COMPARISON OF THE STRENGTHS OF 4-INCH SAND-MORTAR CUBES FROZEN AT DIFFERENT AGES. (RATH)

AGE AT		CRUSHING STRENGTH IN LB. PER SQ. IN. FOR MIXES.			PER CENT OF NORMAL STRENGTH.		
Freezing. Days.	Testing. Days.	1 : 2	1 : 3	1 : 5	1 : 2	1 : 3	1 : 5
2	22	1837	1193	657	91	100	91
3	21	1590	1102	526	74	94	80
4	18	1420	792	332	76	73	53
8	16	1537	788	372	85	77	64
15	17	1630	933	477	89	89	78

In Table 19 are given the results of tests by W. C. F. Rath \* on mortars made from various proportions of Medusa cement and pit sand. The purpose of these tests was to determine the effect of age at freezing on strength. The mortars were of plastic consistency. Specimens were allowed to set in air at room temperature before freezing; they remained frozen one day and were then allowed to cure in the air of the laboratory until tested. The data show that under this treatment the maximum injury to the compressive strength of mortars three weeks old was produced when they had set for three or four days before freezing. It is probable that this effect would have been less marked if the specimens had been allowed to age for two or three months after freezing.

The effect of freezing for varying lengths of time immediately after making is illus-

trated in Fig. 18. The ratios of the lengths of the ordinates to the two curves for the same mix furnish a comparison between the strength of

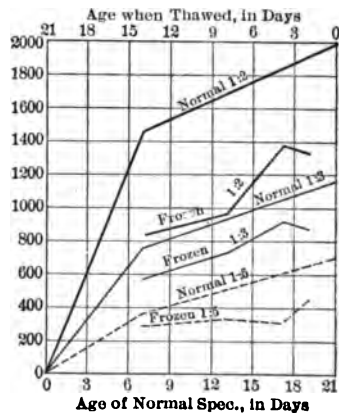


FIG. 18.—The Effect of the Length of Freezing Period on the Compressive Strength of 4-inch Sand-mortar Cubes.

\* Thesis, University of Wisconsin, 1906.



mortars normally cured and the strength attained in an equal period after thawing. Evidently freezing prior to setting more seriously affects the strength of mortar or concrete than freezing after setting.

Table 20 \* shows the effect of freezing on the strength of 1 : 2 : 4 concrete both before and after initial set has taken place. As would be

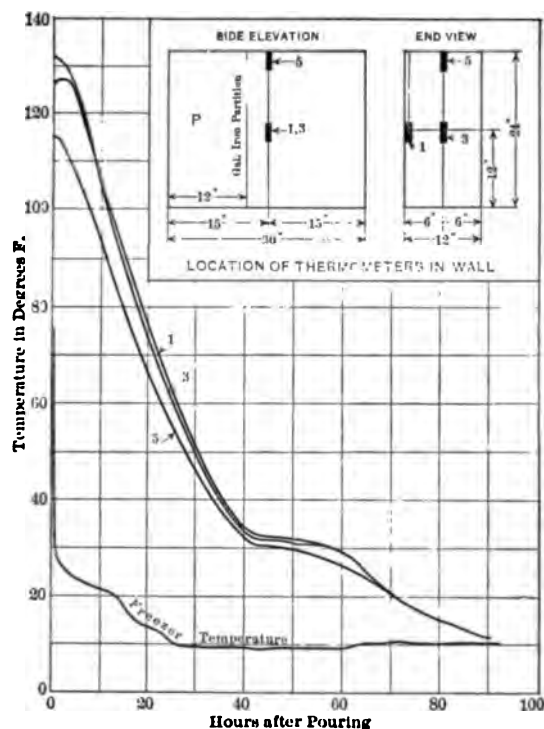


FIG. 19.—Diagram Showing the Rate of Cooling for a 12-inch Wall of 1 : 2 : 4 Gravel Concrete. Consistency was medium.

strength is greatest when the intervening thawing periods are from four to six days long.

Experimental evidence further indicates that lean mixtures suffer less loss in strength by freezing than rich mixtures. Concrete of dry consistency resists freezing better than one of wet consistency. After frozen concrete or mortar has been thawed it should be kept wet for three or four weeks in order that it may harden properly.

**540. The Rate of Cooling of Concrete Setting at Low Temperatures.**—The change in temperature at different points in a concrete wall sub-

\* Compiled from results of tests made by classes electing work in concrete at the University of Wisconsin.

expected, the effects on the seven-day strengths are much more severe than upon the two-month strengths. After final thawing these specimens were sprinkled twice a day until tested. The tests also show that the final strength of Portland cement concrete of wet consistency is not destroyed by a small number of freezings before or after setting, nor by a small number of alternate freezings and thawings, that the magnitude of the effects of such treatment varies with the cement. Other freezing tests by Rath on plastic mortars show that the effect from alternate freezing and thawing is greater than from freezing once, and that the effect of such alternations on the

TABLE 20.—EFFECT OF FREEZING ON THE COMPRESSIVE STRENGTH OF 1 : 2 : 4 PORTLAND CEMENT CONCRETE

All concrete was made of Universal (U) or Alpha (A) cement, good pit sand, and broken limestone passing a 14-in. mesh. The consistency was wet, not sloppy. The 6×18-in. cylinders, used as specimens, were made in iron molds.

Year Made.	No. of Tests.	Brand of Cement.	AGE WHEN		ALTERNATELY FROZEN AND THAWED.		Time Sprinkled Twice a Day, in Days.	Age at Test, in Days.	Ave. Temp. in Refr. °F.	Compressive Strength, Lb. per Sq. In.
			Placed in Refrigerator in Hours.	Removed from Refrigerator, in Days.	No. Cycles	Total Time, in Days.				
Specimens Made with Normal Water.										
12	2	U	0	7	0	0	0	7	16	114
13	2	A	0	7	0	0	0	7	8	0
12	2	U	6	7	0	0	0	7	16	167
15	2	U	6	6	0	0	0	7	10	20
13	2	A	6	7	0	0	0	7	8	0
12	2	U	18	6	0	0	0	7	15	284
13	2	A	17	6	0	0	0	7	7	267
15	2	U	30	5	0	0	0	7	12	219
13	2	A	28	6	0	0	0	7	7	310
12	2	U	66	4	0	0	0	7	12	797
15	2	U	54	4	0	0	0	7	15	440
13	2	A	67	4	0	0	0	7	7	645
15	2	U	78	3	0	0	0	7	16	684
12	2	U	Not Frozen				7	7		1146
15	2	U					7	7		862
13	2	A					7	7		830
12	2	U	0	7	0	0	70	77	16	2775
15	4	U	0	6	0	0	57	63	10	985
13	2	A	0	7	0	0	56	63	8	970
12	2	U	0		6	7	70	77	16	2605
15	2	U	0		7	7	56	63	11	870
13	2	A	0		3	7	56	63	8	1200
12	2	U	Not Frozen				77	77		2370
15	2	U					63	63		2323
13	2	A					63	63		1460
Specimens Made with a Seven Per Cent Salt Solution.										
12	2	U	0	7	0	0	0	7	16	375
15	2	U	0	6	0	0	0	7	10	155
13	2	A	0	7	0	0	0	7	8	124
12	2	U	0	7	0	0	70	77	16	2284
15	2	U	0	6	0	0	56	63	10	678
13	2	A	0	7	0	0	56	63	8	900
12	2	U	Not Frozen				77	77		2055
15	2	U					63	63		1625
13	2	A					63	63		800

jected to freezing temperature while setting is shown in Fig. 19.\* The forms of this wall were pine boards,  $1\frac{1}{2}$  in. thick. A cover of the same thickness was placed on top of the wall immediately after pouring. Temperatures were measured by resistance thermometers connected to a Callendar recorder. This wall was withdrawn from the freezer when four days old and the  $12 \times 12 \times 24$ -in. prism (*P*) tested at six days. The strength of the prism was 415 lb. per sq.in. or 65 per cent of the strength of concrete from the same batch cured for six days in the molds at room temperature. From a number of tests like the above, in which different cements and walls 6 and 12 in. thick were used, it appears that concrete of medium consistency placed in 2-in. plank forms at a temperature of 80 to 100° F. will set before freezing providing the wall is at least 6 in. thick and the outside temperature is not less than 0° F. Overheating of the materials should be avoided, since in the above tests heating to 150° F. caused a decrease in the normal strength.

**541. The Effect of Alternate Freezing and Thawing on Hardened Concrete and Mortar.**—To test the resistance of concrete to frost, specimens are sometimes subjected to alternate periods of freezing and thawing. The lower temperature is often about 10° or 15° and the thawing is commonly done by plunging into the water at 150° to 200° F. Ten alternations under such conditions should not lower the crushing strength of good concrete or mortar more than 30 per cent. From tests on mortars and concretes at the University of Wisconsin it appears that rich mixtures suffer a greater proportionate loss in strength under the alternate freezing and thawing test than lean mixtures; also wet mixes suffer more than dry ones.

**542. The Effect of Adulterants in Lowering the Freezing Point.**—Small percentages of salt or calcium chloride dissolved in the mixing water serve to lower the freezing point of the mix and thereby permit hardening at temperatures lower than 32° F. With some cements the use of small percentages of salt appears to diminish the strength; with others the reverse is true. So far as published results show, the use of a 10 or 12 per cent (by weight) solution of common salt has rarely decreased the long-time strength over 25 or 30 per cent, and in most cases the weakening in final strength is much less. The use of salt in reinforced concrete should be prohibited, since in damp locations it may cause rusting of the steel. A common rule for the use of salt is: Add 1 per cent of salt to the mixing water for each degree Fahrenheit below 32.

Tests by H. E. Pulver and S. E. Johnson † at the University of Wisconsin, on the effects of calcium and sodium chloride solutions on the

\* From a thesis by Messrs. O. A. Bailey and F. D. Bickel at the University of Wisconsin, 1915.

† *Concrete Cement Age*, Vol. 3, p. 256.

TABLE 21.—THE EFFECT OF CALCIUM CHLORIDE AND SODIUM CHLORIDE ON THE COMPRESSIVE STRENGTH OF 1 : 2 : 4 CONCRETE

Materials:—Atlas cement, good pit sand weighing 108 lb. per cu.ft., and limestone passing a 1½-in. mesh and weighing 90 lb. per cu.ft.

Consistency:—Wet.

Storage:—Normal specimens stored in air one day, in water 13 days. Low-temperature specimens, placed out of doors or in a refrigerator immediately after molding, remaining frozen until tested.

Testing:—Each result represents tests on 4-in. cubes. Specimens were embedded in blotting paper on a spherical bearing block.

PER CENT SALT.		SPECIMENS CURED AT ROOM TEMP.				SPECIMENS CURED AT LOW TEMP.			
CaCl <sub>2</sub>	NaCl	Temp. of Batch at Mixing, ° F.	Compressive Strength in Lb. per Sq.In. at			Temp. (in °F.) at Mixing.		Compressive Strength in Lb. per Sq.In. at	
			14 Days.	60 Days.	360 Days.	Batch.	Out of Doors.	14 Days.	60 Days
0	0	52	1910	3010	3580	52	13	213	427
0	6	52	1684	2620	2895	51	13	482	685
0	9	65	1525	2385	3055	51	13	680	942
0	12	68	1270	2060	2485	51	13	813	1192
0	15	58	1335	2220	2740	42	13	614	1060
2	0	59	1920	3220	3740	41	17	420	466
4	0	60	2105	3510	3880	44	17	444	564
6	0	60	1725	3280	3670	55	18	349*	367*
8	0	61	1510	3070	3155	46	1	286*	334*
10	0	61	1655	3025	3330	46	1	234*	348*
2	6	59	1600	2650	3150	52	15	817	992
2	9	63	1695	2590	3100	52	15	848	1185
2	12	64	1420	2440	2805	41	7	690	1045
2	15	60	1320	2350	2725	38	7	583	801
4	6	56	1685	2550	2960	52	21	785	914
4	9	58	1550	2390	2965	51	20	755	926
4	12	58	1710	2935	3680	45	15	766	1215
4	15	55	1245	1880	2410	45	15	713	1200
6	6	60	1310	2475	2575	43	20	680	988
6	9	54	1305	2370	3025	38	20	480	850
6	12	54	1345	2420	.....	52	21	505	615
6	15	52	1380	2415	.....	52	21	527	863
8	6	60	1120	2075	2490	51	21	390	580†
8	9	58	1135	2085	2445	50	30	487	654
8	12	60	1110	1995	2525	52	30	402	589
8	15	59	1550	2605	2940	44	14	535	658†

\* Badly disintegrated.

† Edges were spalled to some extent.

TABLE 22.—RESULTS OF COMPRESSION TESTS MADE ON PORTIONS OF PORTLAND CEMENT MORTAR BUILDING BLOCKS TAKEN FROM WALLS SUBJECTED TO A TEMPERATURE OF 900° C. FOR ONE HOUR

Wall No.	Kind of Block.	Consistency.	CRUSHING STRENGTH IN LB. PER SQ. IN.					
			From Exposed Face Proportions.			From Unexposed Face Proportions.		
			1 : 2	1 : 4	1 : 8	1 : 2	1 : 4	1 : 8
1	{ 1 piece, double air-space {	Damp	885	752	....	1709	1140	....
		Medium	1328	831	462	2182	1415	....
		Wet	880	897	376	1891	1692	....
3	{ 1 piece, double air-space {	Damp	1165	552	493	1874	909	....
		Medium	1507	1155	464	2694	1932	....
		Wet	1957	918	....	2477	1494	....
2	{ 1 piece, single air-space {	Damp	1016	653	401	1650	714	....
		Medium	971	812	482	1530	1078	....
		Wet	1949	779	482	2391	1088	....
4	{ 1 piece, single air-space {	Damp	1422	....	512	2181	....	....
		Medium	1846	1205	617	2309	1484	....
		Wet	1843	1240	....	2310	1518	....
5	{ 1 piece, single air-space {	Damp	1034	466	408	1773	987	....
		Medium	1482	774	665	2047	843	....
		Wet	2116	1542	671	2518	2067	....
6	{ 1 piece, single air-space {	Damp	1141	505	....	1189	663	....
		Medium	1550	922	....	1502	1044	....
		Wet	2215	1163	....	2397	1108	....
7	{ 1 piece, single air-space {	Damp	1153	578	502	1411	615	....
		Medium	1370	974	509	1548	1159	....
		Wet	2143	928	521	2467	1043	....
8	{ 1 piece, single-air-space {	Damp	1202	792	307	1208	841	....
		Medium	1812	800	495	2628	837	....
		Wet	1873	1204	539	3307	1922	....
9	{ 2 piece {	Damp	952	350	....	1242	669	....
		Medium	1201	802	552	1358	932	....
		Wet	1244	992	....	1345	1223	....
10	{ 2 piece {	Damp	464	....	472	885	....	....
		Medium	663	348	....	1253	1132	....
		Wet	788	622	398	1503	1329	....

\* In most cases values in the table represent four or more tests of pieces from three blocks.

strength of 1 : 2 : 4 concrete subjected to freezing conditions, are abstracted in Table 21. The combination of 2 per cent calcium chloride with 9 per cent sodium chloride gave the best results of any of the

solutions under freezing temperatures. Since calcium chloride alone greatly hastens the set, its employment in the field will require very rapid handling during pouring.

**543. Resistance of Concrete and Mortar to High Temperatures.—**

Observations after conflagrations like the San Francisco, Baltimore and Chelsea fires have shown that concrete possesses a high resistance to fire. Many examples have been cited of concrete buildings which were left standing alone in fire-swept areas of these cities.

Tests reported by R. L. Humphrey, in *Bulletin* No. 370, *U. S. G. S.* show that mortar blocks and concrete beams have equal, if not superior, resistance to fire of any of the building materials employed for fire protection. The tests were made on wall panels at the Underwriters' Laboratories in Chicago. Specimens were well cured and two months or more old. The temperature of the furnace was gradually raised to 800° C. in one-half hour and maintained between 800 and 1000° C. for one and one-half hours. Careful records of temperature of the furnace, front and rear of the wall were obtained by means of pyrometers. Immediately after the panel was withdrawn from the furnace, the face exposed to the fire was soaked with a hose. A summary of the effects of this treatment on the strength of portions of mortar blocks taken from both sides of the walls is given in Table 22. These tests show that the resistance of mortars to fire increases with the richness of the mix for proportions between 1 : 2 and 1 : 8, and that generally wet mixtures stand a high temperature slightly better than medium mixes and much better than dry mixes.

Humphrey's fire tests on 1 : 2 : 4 concrete from the plain beams used in the tests tabulated in Table 8 indicate no marked difference in the resistance of the granite, limestone, and gravel concretes. The cinder concrete showed less conductivity but it suffered greater disintegration than the above-mentioned concretes. This was due to the softness of the cinders and the presence of 24.5 per cent. of combustible material.

Obviously, the resistance to fire of a wall heated on one face only will be much greater than the resistance of a small cube heated on all sides, yet tests on the latter type of specimen afford information on the relative resistance of various kinds of concrete. In Table 23 have been placed results showing the effect of fire upon the compressive strength of concrete. These were gotten by I. H. Woolson.\* For these tests the furnace was heated at a rate of 500° F. in forty-five minutes. Specimens were held at the temperature indicated for ten minutes.

Prof. Woolson found that the modulus of elasticity for both trap and limestone concrete was reduced over 60 per cent by heating to 500° F. In other tests † on 1 : 2 : 3 concrete heated to 1500° F. in forty-five min-

\* *Proc. A. S. T. M.*, Vol. 5, 335.

† *Proc. A. S. T. M.*, Vol. 7, p. 404.

TABLE 23.—EFFECT OF FIRE ON THE COMPRESSIVE STRENGTHS OF 4-INCH CONCRETE CUBES. MADE OF TRAP ROCK AND LIMESTONE

Concrete was of medium consistency and consisted of 1 part mixed Portland cement 2 parts sand and 4 parts  $\frac{1}{4}$ -in. broken stone. Age of specimens four to six weeks. Each value represents three tests.

TIME BETWEEN HEATING AND TESTING, DAYS.		Temper- ature of Furnace °F.	COMPRESSIVE STRENGTH IN LB. PER SQ. IN.		CONDITION AFTER HEATING.	
Trap.	Lime- stone.		Trap.	Lime- stone.	Trap.	Limestone.
2	3	0	1903	1817		
2	3	500	1920	1234	.....	Slightly brittle
2	3	750	1840	1244	Slight cracks	Brittle
2	3	1000	1410	1043	Brittle, cracks	Stone partly calcined
2	3	1250	1244	973	Brittle, cracks	Calcined throughout
10	3	1500	1556	765	Sound	Full of cracks
10	3	1750	923	813	Full of cracks	Fragile
9	3	2000	847	.....	Full of cracks	} Crumbled on cooling
		2250	501	.....	Partly fused	

utes and maintained at that temperature for fifteen minutes he proved that the modulus of elasticity may be reduced to one-tenth of its normal value. The latter experiments were made on trap, quartz gravel, and cinder concretes and showed that the relative loss in stiffness was least for the cinder concrete and most for the quartz gravel.

In general, it may be stated that concrete heated above 1000° F. will lose a large portion of its strength and stiffness. If it is then exposed to the air it will show marked surface disintegration; in small specimens the entire structure may disintegrate in the course of two or three weeks. Such disintegration is very pronounced in concretes made from limestone or dolomitic aggregates. Lean mixes suffer disintegration more quickly than rich ones. On the other hand, if the specimens are wet while hot, an arrest of the disintegrating influences is brought about.

**544. The Coefficient of Expansion of Concrete and Mortar.**—Table 24 contains values of the linear coefficient of thermal expansion of concrete for atmospheric ranges of temperature. For mixes commonly used an average value is 0.000006 per ° F., which is approximately the same as the coefficient of expansion for iron and carbon steel.

**545. Other Thermal Properties.**—Tests by C. L. Norton \* on the specific heat of various mixtures of concrete gave the results in Table 25. In these tests he employed Regnault's method of mixtures, using an ordinary double calorimeter.

\* *Proc. Nat. Asso. Cement Users*, Vol. 7, p. 78.

TABLE 24.—LINEAR COEFFICIENT OF EXPANSION OF CONCRETE AND MORTAR AT NORMAL TEMPERATURES

Mix.	Aggregate.	Coefficient per °F.	Authority.	Reference.
1 : 0	.....	.0000070	Keller	} <i>Tonind'z't'g</i> , No. 24, 1904
1 : 2	Pit gravel	.0000056	Keller	
1 : 4	Pit gravel	.0000058	Keller	
1 : 8	Pit gravel	.0000053	Keller	
1 : 0	.....	.0000059	Bonniceau	} <i>Ann. Ponts Chaus.</i> 1863, p. 181
1 : 2	Sand	.0000066	Bonniceau	
1 : 2 : 4	Sand and limestone	.0000055	Pence	} <i>Jour.W. S. E.</i> , Vol. 6, p. 549
1 : 2 : 4	Sand and gravel	.0000054	Pence	
1 : 5	Pit gravel	.0000053	Pence	
1 : 2 : 5	Sand and gravel	.0000066	Hallock	} Reid, p. 171
1 : 2	Sand	.0000056	Hallock	

TABLE 25.—THE SPECIFIC HEAT OF CONCRETE. (NORTON)

Range of Temp. ° F.	1 : 2 : 5 Stone Con- crete.	1 : 2 : 4 Stone Con- crete.	1 : 2 : 4 Cinder Con- crete.
72 to 212	0.156	0.154	
72 to 372	0.192	0.190	0.180
72 to 1172	0.201	0.210	0.206
72 to 1472	0.219	0.214	0.218

TABLE 26.—THE COEFFICIENT OF THERMAL CONDUCTIVITY OF CON-  
CRETE. (NORTON)

Temperature of Hot Side of Plate in Degrees.		Mixture.	Coefficient in Calories per 1 Deg. C. per Sq. Cm. per Cm. per Sec.	Coefficient In B.T.U. per 1 Deg. F. per Sq. Ft. per In. Thick per 24 Hr.
C.	F.			
35	95	Stone 1 : 2 : 5	0.00216	150
50	122	Stone 1 : 2 : 4 not tamped	0.00110 to 0.00160	76 to 114
50	122	Cinder 1 : 2 : 4	0.00081	56
200	392	Stone 1 : 2 : 4	0.0021	146
400	752	Stone 1 : 2 : 4	0.0022	153
500	932	Stone 1 : 2 : 4	0.0023	160
1000	1832	Stone 1 : 2 : 4	0.0027	188
1100	2012	Stone 1 : 2 : 4	0.0029	202



Norton also determined the coefficient of thermal conductivity ( $K$ ) for concrete.

$$K = \frac{Qd}{(t_1 - t_2)As}$$

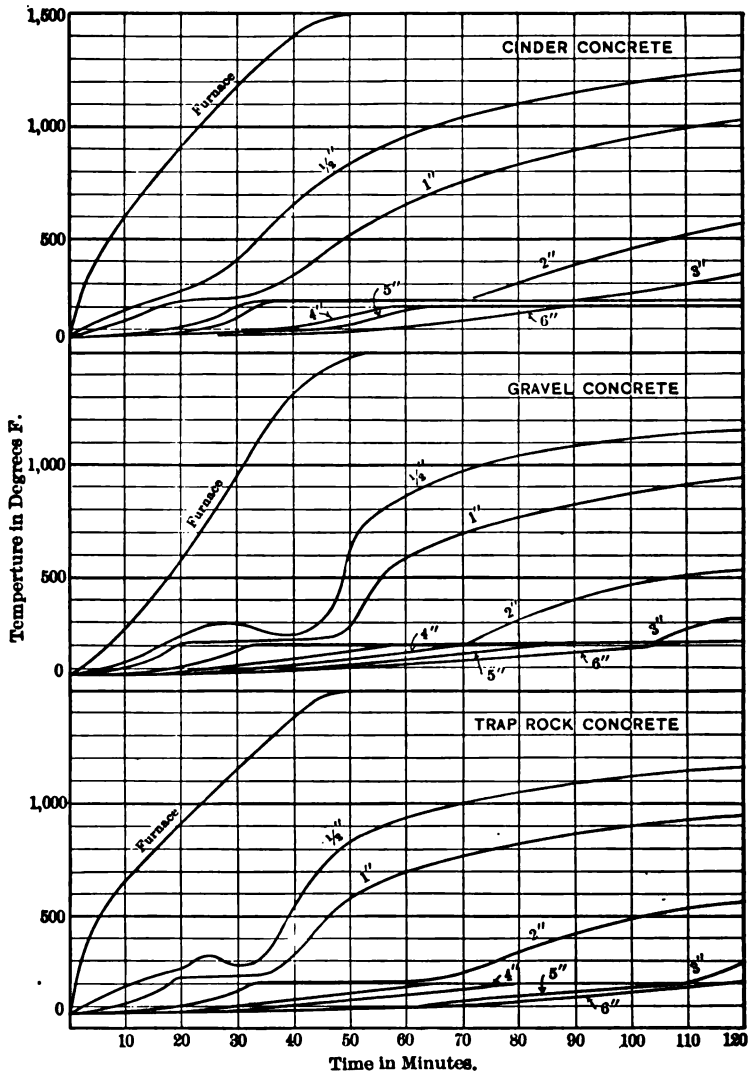


FIG. 20.—The Thermal Conductivity of Concrete. (Woolson, *Proc. A.S.T.M.*, Vol. 7, p. 406.)

in which  $Q$  = the quantity of heat flowing through a plate whose area =  $A$  and thickness =  $d$ ,  $s$  is the time of flow, and  $t_1$  and  $t_2$  the temperatures

on the hotter and cooler sides of the plate, respectively. His results for concrete, together with the values for other building materials, appear in Table 26. It will be noted that the conductivity of cinder concrete is only about 40 per cent that of stone concrete.

Although concrete is superior to the metals and natural stones, it must not be considered a first-class insulating material. At room temperatures it is greatly surpassed as a heat insulator by asbestos, mill shavings, powdered magnesia, mineral wool, hair felt, pulverized cork, and hair cloth; and at high temperatures it is inferior to such substances as asbestos, infusorial earth, and powdered magnesia. The protective value of concrete lies in a high resistance to fire coupled with a fairly low conductivity and high strength.

Woolson also determined the rate of flow of heat into different classes of concrete when the exposed face of the block was subjected to a temperature of 1500° F. His measurements were made by Le Chatelier pyrometers placed at distances of  $\frac{1}{2}$ , 2, 3, 4, 5, 6, and 7 in. from the exposed faces of rectangular prisms 8 in. deep and  $7\frac{1}{4} \times 13\frac{1}{2}$  in. in elevation. After one hour at the above temperature the records obtained on single specimens of 1 : 2 : 5 cinder, 1 : 2 : 4 gravel, and 1 : 2 : 4 trap rock concrete are shown in Fig. 20.

In certain tests Woolson embedded  $\frac{3}{4}$ -in. bars in the axes of 8×8×36-in. concrete blocks. These blocks were of similar proportions and ingredients to those mentioned above. The bars were cut longer than the blocks and projected from either 8×8-in. face. One 8×8-in. face was gradually heated to 1700° F. and measurements of the temperature at different points along the bar were made. After one hour at the above temperature a point on a bar 2 in. from the exposed face of the prism had a temperature of only 700° to 1000° F.; a point 5 in. from the surface reached 400° F. to 500° F.; while a point 8 in. from the surface attained the temperature of boiling water. These tests and others indicate that 2 in. of good concrete affords ample protection to embedded steel against fire.

In the previously mentioned tests conducted by Humphrey, the maximum temperature at the outside surface of the 8-in. walls, made of concrete beams, varied from 58° C. for the granite to 43° C. for the cinder concrete. This was obtained after the inside temperature had been held at 800–1000° C. for about one and one-half hours.\*

A few experiments † on hollow concrete cylinders 36 in. inside diameter, 36 in. high and 6 to 8 in. thick, showed that the temperature gradient

\* For the calculation of the rate of flow of heat into concrete and other materials reference may be made to Ingersoll and Zobel's *The Mathematical Theory of Heat Conduction, with Engineering and Geological Applications*; also *Engr. News*, Vol. 70, p. 866.

† Thesis by Melin and Pulver, University of Wisconsin, 1910.

through the walls approximately a straight line after the internal temperature had been maintained constant at 700° F. and the outside at room temperature for a couple of days. A 1 : 2 : 4 concrete made of limestone was used.

### THE DURABILITY OF CONCRETE

**546. Effect of Sea Water.**—Experience with concrete and mortar exposed to the action of salt water has resulted in conflicting evidence regarding its durability. Exhaustive tests extending over a long period of time giving full account of the materials used and the exact conditions of manufacture are greatly needed.

Probably the most comprehensive series of tests on mortars and concretes cured in sea water are those of the Society of Scandinavian Portland Cement Manufacturers. These experiments were begun in 1896.\* They comprise tests on 3500 mortar cubes each 2.8 in. on an edge and over 100 blocks each about 1 cubic yard in volume. The majority of the test-pieces were made with three Scandinavian and one English Portland cement, and with Teil hydraulic lime from France. Mortar mixtures were neat 1 : 1, 1 : 2, and 1 : 3, the proportions of the concrete blocks were 1 : 1 : 2 $\frac{1}{8}$ , 1 : 2 : 4 and 1 : 3 : 5 $\frac{1}{8}$ . The sand used in the concrete blocks was too fine for best results. Both mortar and concrete specimens were cured with fresh water before immersing in the sea. Mortar test-pieces were immersed at five different localities, in three of which the conditions varied as follows: At Vardo, in the extreme north of Norway, the average annual temperature is 32° F., the per cent salt in the water is 3 $\frac{1}{2}$ ; at Esbjerg, in the extreme southeast of Denmark, the average annual temperature is 45° F., the strength of the salt water is the same as at Vardo; at Degerhamn, on the southeast coast of Sweden, the proportion of salt in the water is only 0.5 per cent. Specimens were submerged both below low water and at midtide height.

From Poulsen's report of observations on the concrete blocks and the strength after ten years in salt water, the following conclusions appear to be justified:

1. Chemical action of sea water alone does not cause destruction of Portland cement mortars and concretes.
2. Climatic conditions, principally the mechanical action of the tides and frost, are chiefly responsible for the disintegration of concrete and mortar mixtures in sea water. The hydraulic lime from Teil was especially adversely affected by the midtide treatment at Vardo.
3. Mixtures containing mortars leaner than 1 : 2 should not be used in maritime construction.

\* *Cement in Sea Water*, A. Poulsen, Copenhagen; also *Proc. I. A. T. M.*, 5th Congress, XI.

4. From the experiments on artificially graded mixtures, sands for marine concrete work should contain from  $\frac{1}{3}$  to  $\frac{2}{3}$  of particles passing a No. 30 sieve.

5. The addition of small percentages of pulverized trass or very finely ground sand to the cement is advantageous.

A valuable set of experiments on the effect of sea water on concrete was begun in 1909 by the Aberthaw Construction Company of Boston.\* Twenty-four piers 16 ft.  $\times$  16 in.  $\times$  16 in. are suspended from a wharf in Boston harbor so that both the effect of continued submergence below low water and the action of the tides can be observed. Crushed trap rock 98.25 per cent of which was between  $1\frac{1}{2}$  and  $\frac{1}{4}$  in. diameter and a good quality of bank sand, 52 per cent of which passed a No. 30 sieve, were the aggregates.

Portland cements of high, medium, and low alumina contents, iron ore cement, and slag Portland cement were used. One pier of 1 : 3 : 6 proportions was made with 10 per cent of the cement replaced by hydrated lime, another with 5 per cent of pulverized clay, and in a third, waterproofing with a soap and alum solution was tried. The mixes were 1 : 1 : 2, 1 :  $2\frac{1}{2}$  :  $4\frac{1}{2}$  and 1 : 3 : 6, most of the concrete was of quaking consistency, although wet and dry mixes were also made. The specimens were well cured before submergence. After five years a careful examination of the piers indicated: 1, In the untreated specimens which were eroded on the surface most of the action occurred between the high and low water-marks; 2, the erosion in such cases was most pronounced on the faces and sides of the specimens; 3, evidence of the superiority of any one type of Portland cement is not furnished by a five-year exposure. 4, mixes containing hydrated lime or soap and alum were badly attacked throughout their submerged length, the pulverized clay mix was in fair condition; 1 : 3 : 6 mixes are too lean even when very well mixed and of proper consistency; 5, a wet or a quaking consistency is much superior to a dry consistency, the 1 : 1 : 2 mixtures of wet consistency having withstood the attack very well.

Chemical tests on fragments of large concrete blocks which had been immersed in the Baltic Sea for thirteen years are reported by W. Czarowski.† These blocks were made of 1 part cement,  $2\frac{1}{2}$  parts sea sand, and approximately 7 parts rubble and gravel mixed in 3 : 1 proportions respectively. Slag, Portland and sand-blended cements were used. In both Portland and sand-blended cements the alumina content was normal, being about  $6\frac{1}{2}$  per cent. The outside surfaces of blocks were in good condition, but when they were blown open white exudations were found. Chemical analysis of fragments of these blocks showed that a liberation of

\* *Engr. Record*, Vol. 69, p. 344.

† *Proc. I. A. T. M.*, 6th Congress, XVII, 1.

lime and absorption of magnesia and sulphur trioxide had taken place in the cement. The mortar of granite masonry blocks submerged for seven years at the same place showed less deterioration than the concrete.

Lombard and Deforge \* report results of several sets of tests on different kinds of cement subjected to the action of sea water at La Rochelle, France. Two neat blocks of an English Portland cement and one of a slow-setting, low-alumina natural cement showed very little evidence of deterioration after more than fifty years exposure. Jasson and Porte de France Portland cement blocks after twenty-nine years were in very good condition. The slight erosion probably being due to wave action. Teil lime blocks, however, disintegrated in fifteen to nineteen years.

Tests by the U. S. Bureau of Standards † covering an immersion period of one or two years show that concrete made of natural, slag, iron ore and typical American Portland cements does not suffer disintegration and increases in strength.

Messrs. R. J. Wig and L. R. Ferguson have written a very important report, ‡ based on examinations of a large number of structures situated along the coasts of continental United States. The results of their investigations may be summarized as follows:

1. Well-made plain concrete is permanent in sea water provided it is properly placed and protected from erosion and abrasion. Plain concrete structures in northern latitudes are exposed to much more severe conditions than those built on the southern coasts.

2. For concrete exposed to sea water, the chemical composition of the cement is immaterial provided it passes standard requirements. The aggregate should be composed of tough particles so graded as to produce a hard, dense concrete, and enough water should be used to produce a plastic mix. Excess water is very harmful. Sea water can be used in gauging plain concrete, but should not be used when the work is reinforced and exposed to the air. In placing, great care must be taken to make the forms tight. Seams and porous spots must be avoided. If subjected to erosion or abrasion, surfaces above the low-water line must be protected. Walls and piers may be faced with stone. Piles may be protected by wooden fenders.

3. Reinforced concrete, when used above mean-tide level, is likely to disintegrate through corrosion of the reinforcement unless the latter is galvanized or protected by a considerably thicker shell of concrete than is now the current practice. Corrosion of reinforcement proceeds more rapidly in structures of the southern coasts than in those farther north where the water is colder.

\* *Proc. I. A. T. M.*, 6th Congress, XVII.

† *Technologic Paper* No. 12, by Bates, Phillips, and Wig.

‡ *Engr. News-Record*, Vol. 79, pp. 532, 641, 689, 737, and 794.

**547. Effect of Alkali Water.**—In regions where alkali is present in the soil and ground waters, trouble has been experienced due to the disintegration of concrete and masonry structures. Water running through such soil may contain as much as 1 per cent of alkali salts, the most destructive of which are the sulphates of magnesium and sodium, and the carbonate of sodium. Examinations of structures show that the most pronounced action occurs in portions of the masonry which are subjected to alternate wetting by the alkali solutions and drying by the air. Laboratory tests have shown that alkali salt crystals, which form very slowly, if at all, during the period of immersion, grow rapidly when the degree of saturation of the solution is increased by desiccation, and cause stress in the surrounding structure. If such crystals find lodgment in the surface of a porous substance, their expansion will break off portions of the material and gradually cause disintegration. As previously noted, such action is also partially responsible for the surface disintegration of concrete in sea water. When concrete or mortar is continuously submerged in alkali water the action is not marked.

If possible, alkali water should be drained away from concrete foundations, footings and similar structures. When concrete is used as a conduit for such waters every precaution must be taken to secure an impervious mix with a very dense surface. Especial care must be observed in curing, since it has been shown in Art. 529 that improper curing may ruin work which would otherwise be water-tight.

Many examples of the destructive action of alkali waters are cited in *Bulletin* No. 69, of Montana Agricultural College; *Bulletin* No. 132, of Colorado State Agricultural College, *Trans. A. S. C. E.*, Vol. 67, p. 572; *Technologic Paper* No. 12, U. S. Bureau of Standards, which contains a good bibliography of the subject; and in *Technologic Paper* No. 44.

**548. The Effect of Sewage on Concrete.**—Whether concrete or mortar can be used in sewer constructions is dependent upon the character of the sewage. There are concrete sewers which have been in successful operation for many years, but there are also published accounts of others which have failed. If the sewage is of such nature that a strong odor of hydrogen sulphide is evolved, sulphuric acid will be formed with sufficient strength to attack the lime compounds of the concrete and produce disintegration.

Messrs. Barr and Buchanan \* assert that the hydrogen sulphide is formed both by the bacterial decomposition of the sulphur containing proteins, and related compounds, and by the reduction of sulphates in the water supply. They found that the escaping hydrogen sulphide which is dissolved by the moisture on the walls above the sewage is oxidized, not alone by the air, but also by bacterial action. Examinations of septic tanks by these investigators showed that a soluble sulphur ( $\text{SO}_4$ ) content in

\* *Bulletin* No. 26, Iowa State College.

the raw sewage equal to 427 parts per million was sufficient to produce disintegration in a concrete dosing chamber. The disintegration of concrete sewers due to the above action always takes place above the water level. In the septic tanks investigated the action was particularly noticeable on the insides of the roofs.

A detailed report of action in an English sewage system similar to that observed by Barr and Buchanan may be found in an article by S. H. Chambers, *Concrete and Construction Engr.*, Vol. 5, p. 348, or *Engr. Rec.*, Vol. 61, p. 633.

**549. Electrolysis of Concrete.**—The disintegrations of certain reinforced concrete structures have apparently been due to electrolytic action of stray currents from neighboring power circuits. Studies of such action have been made by a number of investigators all of whom report that, under certain conditions, the integrity of a structure may be endangered. Probably the most exhaustive study of electrolysis in concrete has been made by the U. S. Bureau of Standards; its report is rendered in *Technologic Paper No. 18*, by Rosa, McCullum and Peters. In these experiments over four hundred specimens of the type shown in Fig. 21 were

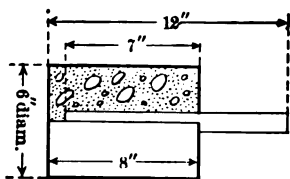


FIG. 21.—Type of Specimen Used in Electrolysis Tests.

tested under voltages varying from 0 to 70. The test-pieces were made of 1 :  $2\frac{1}{2}$  : 4 concrete of Portland cement, sand, and broken stone. They were cured under damp sand after removal from molds for a period of twenty days, then they were placed in a damp closet until tested. The age of specimens when the tests were begun varied from a few days to about one year. The voltage was maintained

continuously on some specimens for a period of over one year. Several different brands of cement were used, but the results showed no great difference in efficiency of corrosion for the normal American Portland cements. White Portland cement was inferior to the normal brands in resisting corrosion.

The conclusions of these investigators may be partially summarized as follows:

1. Plain concrete is not affected by stray currents unless power lines are grounded in it.
2. A potential gradient of approximately 60 volts per foot was required to cause destruction of the test-pieces when the current flowed from the steel anode into normal wet concrete. With dry concrete the voltage required to produce disintegration was much higher.
3. The resistance of normal wet concrete rises rapidly with the length of time of passage of the current.
4. The use of salt in the mixing water is to be condemned, since it

increases the conductivity of the concrete and thereby accelerates the corrosion of the reinforcement.

5. The splitting of the concrete surrounding a steel anode is caused by the oxidation of the steel. The volumetric increase of the steel due to oxidation was sufficient in a certain specimen to produce a radial stress of approximately 4700 lb. per square inch.

6. For a given number of ampere-hours and a given temperature, the amount of corrosion is independent of the current strength.

7. When the current flows for some time from the concrete to a steel cathode, the concrete is softened to a depth of  $\frac{1}{4}$  in. or more around the reinforcement and the bond is destroyed.

8. Softening at the cathode occurs under much lower potential gradients than does the anode effect. Since in practice the steel may often be the cathode, danger from such effect is greater than from the anode effect.

9. The passage of the electric current from the concrete into the steel causes a gradual concentration of sodium and potassium near the cathode. When concentrated, the attack of these alkalis is sufficient to soften the concrete and weaken the bond.

10. The danger from stray currents in practice, although well worthy of consideration, has been much overestimated.

11. By encasing concrete which is in contact with water in waterproof membranes, its resistance may be increased and the danger from electrolysis diminished. The electrical resistance of concrete foundations may also be materially increased by surrounding them with a shell of granite masonry.

12. Direct-current power circuits in concrete structures must be insulated from the ground. Pipe lines should be provided with insulating joints where they enter and leave a structure and lead covered cables should be isolated from the concrete.

TABLE 27.—THE SPECIFIC RESISTANCE OF CONCRETE AND MORTAR TO ELECTRICITY. (*Technologic Paper No. 18*)

Proportions.....	Neat	1 : 2	1 : 4	1 : 2 : 3	1 : 2 $\frac{1}{2}$ : 4	1 : 3 : 5	1 : 4 : 7
Resistance, in ohms per cm. <sup>3</sup>	3500	2300	2100	6300	8000	8200	9900

13. When the metal within a building is insulated with respect to metal without, it may well be interconnected but it should never be grounded.

**550. Specific Resistance of Concrete to Electricity.**—Measurements of the specific resistance by the above investigators for mortars and concretes made of river sand and crushed trap rock gave the results in Table



27. The specimens were nine months old and had been soaked for three months previous to the test. Similar tests on granite and limestone rocks showed that limestone has six or eight times, and granite about a hundred times the resistance of the concrete tested. Consequently the character of the aggregate as well as the age, proportions, and moisture content will greatly affect the specific resistance of concrete.

## CHAPTER XV

### PORTLAND CEMENT PRODUCTS

**551. General.**—Within the past quarter of a century building blocks, brick, drain tile and sewer pipe made of Portland cement mortar or concrete have come into general use in construction. The use of these products has been especially large in regions where good aggregates and cement are obtainable at reasonable rates, and in regions where the cost of clay products is large due to freight charges. Furthermore, the wide distribution of suitable aggregates, the small cost of plant equipment, and the possibility of using unskilled labor are factors which have made these industries popular sources of investment for the man with little capital. These factors have had both a good influence and a bad influence on the growth of cement-product industries. They have been of advantage in promoting the widespread use of cement products, but great harm has been done the industries by the large quantity of poor material which has been turned out by incompetent manufacturers.

### CONCRETE BLOCKS AND BRICK

**552. Merits of Concrete Blocks.**—By using hollow blocks it is possible to secure wall constructions which will be fireproof, dampproof and of low heat conductivity without sacrificing the strength of the structure. Concrete blocks can be made true in shape and dimensions and of such size that the maximum efficiency can be secured in laying a wall. In the latter respect they have a decided advantage over brick.

**553. Types of Blocks.**—Fig. 1 shows several types of blocks now on the market. The two-piece block shown in Fig. 1d is held together by a pair of galvanized iron rods  $\frac{1}{4}$ -in. in diameter. Although the one-piece blocks are less expensive to make and lay, yet these advantages are more than offset by the greater degree of imperviousness obtainable in two-piece block constructions.

Attempts have been made to produce blocks with imitation stone faces. Many of these are failures from an esthetic standpoint. A pleasing appearance may be gotten, however, by facing the blocks with a mortar made of crushed stone chips. If colored blocks are required good results may be secured by employing colored sands or screenings. For ornamental work both sand and plaster of Paris molds have been successfully used

**554. Methods of Manufacture.**—Concrete blocks are most economically made from a mixture of fine and coarse aggregate, although many good blocks are made without any aggregate larger than  $\frac{1}{4}$  in. in diameter. If coarse aggregate is used, it should all pass a  $\frac{3}{4}$ -in. mesh and be retained on a  $\frac{1}{2}$ -in. mesh. Gravel is preferable to broken stone because of the greater fluidity and density which it gives to concrete. In other respects the aggregates and cement should conform to the recommendations and specifications found in Ch. XIII and IX, respectively. The proportions commonly used when coarse aggregate is employed vary from 1 : 2 : 3 to 1 : 3 : 5. If mortar alone is used the mixture should not be leaner than 1 : 3 for medium sands and not less than 1 : 4 for the very best quality of

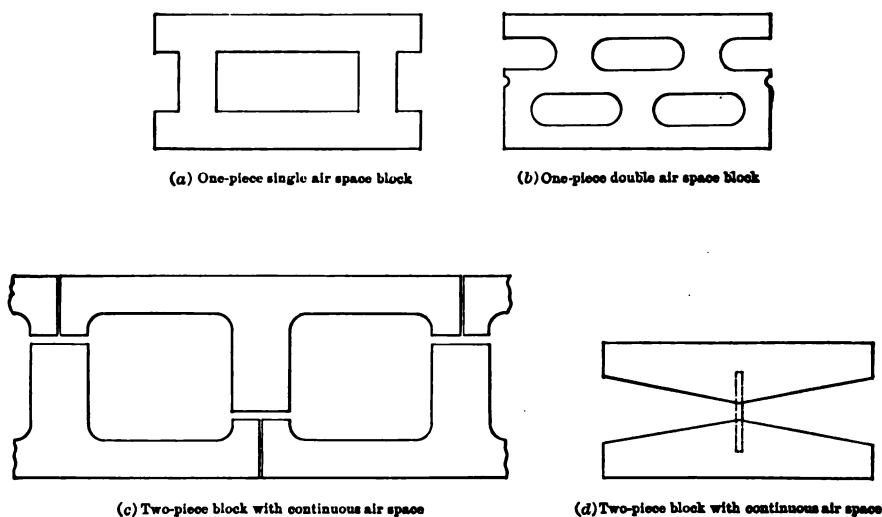


FIG. 1.—Top Views of Different Types of Concrete Blocks.

coarse, well-graded sands. In order to secure maximum strength and density with a minimum use of cement, different proportions should be experimented with in accordance with the methods outlined in Art. 482 and 483. If faced blocks are made, special care should be taken to select a well-graded sand or screenings for the facing mortar. There is a tendency to use too fine sand for this purpose with the result that the blocks are porous and the facing mortar peels off in a short time.

Machine mixing is preferable to hand mixing, but the latter can be successfully used when the number of blocks is small. The essential requisites for thorough mixing are discussed in Art. 488.

Several different methods of tamping are employed to secure dense blocks. The most common method is hand tamping. In large well-

equipped plants pneumatic tampers are considerably used. Molding machines by which pressure is exerted upon the blocks through a toggle joint or by an hydraulic press are also employed. With any of these methods of tamping it is customary to use a dry mealy consistency which will barely moisten under the tamping or pressure which is exerted. Inasmuch as a deficiency in the amount of mixing water will cause a marked increase in the permeability of a block, it is of the utmost importance that all the water which the block can hold and still retain its shape after the mold is removed should be used. The use of a very dry mixture has been responsible for a great many poor blocks.

Blocks of slush consistency are poured into iron molds. By this process it is possible to secure a very dense block with a minimum amount of labor in molding, but it requires a very large outlay in molds since they must remain in place until the blocks have hardened.

No portion of the process of manufacture of concrete products is of more importance than proper curing. Advantageous ways of securing well-cured blocks are discussed in Art. 497.

Blocks should not be placed in a wall until they are capable of bearing a compressive load of 800 lb. per square inch. of superficial area. This may be decreased to 600 lb. per square inch where the blocks are to be used in structures two stories or less in height. Properly made blocks should attain the above strengths in two to three weeks.

**555. The Testing of Blocks.**—There are three rough field tests which can be made to determine the quality of concrete blocks: 1, the ring emitted when struck with a hammer; 2, the rapidity with which a dry block will absorb a bit of water poured upon its face; 3, the ease with which it can be scratched with a knife. A strong dense block will give a clear metallic ring, show little absorption and cannot be cut with a knife.

An excellent test of the imperviousness of a given concrete block wall can be made as follows: A wall four blocks long and four or five courses high is laid in neat cement mortar. A perforated pipe is suspended horizontally a few inches from the face and near the top of the wall. The pipe is then connected with a garden hose and the blocks subjected to a miniature shower. Observations should be made from time to time on the condition of the back of the wall. In tests made at the University of Wisconsin, walls of one-piece dry-process blocks wet through in many instances within an hour or two. Two-piece dry-process blocks and blocks of slush consistency withstood the test for several days.

With laboratory equipment the most important tests are: 1, transverse; 1, crushing; 3, absorption. Fire tests and alternate freezing and thawing tests are also sometimes made. The following methods of testing have been abstracted from the specifications of the *National Association of Cement Users*. (January, 1909.)

## STANDARD METHOD OF TESTING

1. All tests required for approval shall be made in some laboratory of recognized standing, under the supervision of the engineer of the bureau of building inspection or the architect or engineer in charge, or all of these. The manufacturer may be present or represented during said tests, if he so desires. Approval tests are made at the expense of the applicant.

2. For the purposes of the tests, at least twelve (12) samples or test pieces must be provided. Such samples must represent the ordinary commercial product and may be selected from stock by the bureau of building inspection, or in the absence of such bureau, by the architect or engineer in charge.

In cases where the material is made and used in special shapes or forms, too large for testing in the ordinary machines, smaller sized specimens shall be used as may be directed.

3. In addition to the tests required for approval, the weight per cubic foot of the material must also be obtained and recorded.

4. Tests shall be made in series of at least three (3), except that in the fire tests a series of two (four samples) are sufficient.

Transverse tests shall be made on full-sized samples. Half samples may be used for the crushing, freezing and fire tests. The remaining samples are kept in reserve in case duplicate or confirmatory tests be required. All samples must be marked for identification and comparison.

5. The transverse tests shall be made as follows: The samples shall be placed flatwise on two rounded knife-edge bearings set parallel 7 inches apart. A load is then applied on top, midway between the supports, and transmitted through a similar rounded knife-edge, until the sample is ruptured. The modulus of rupture shall then be determined by multiplying the total breaking load in pounds by  $3l$  (three times the distance between supports in inches) and then dividing the result thus obtained by twice the product of the width in inches by the square of the depth in inches.  $R = \frac{3wl}{2bd^2}$ . No

allowance should be made in figuring the modulus of rupture for the hollow spaces.

6. The compression test shall be made as follows: Samples must be cut from blocks, so as to contain a full web section. The sample must be carefully measured, then bedded flatwise in plaster of Paris, to secure a uniform bearing in the testing machine, and crushed. The total breaking load is then divided by the area in compression in square inches, no deduction to be made for hollow spaces; the areas will be considered as the product of the width by the length.

7. The absorption test shall be made as follows: The sample is first thoroughly dried to a constant weight, at not to exceed 212° F. The weight must be carefully recorded. It is then placed in a pan or tray of water, face downward, immersing it to a depth of not less than 2 inches. It is again carefully weighed at the following periods: Thirty minutes, four hours, and forty-eight hours, respectively, from the time of immersion, being replaced in the water in each case as soon as the weight is taken. Its compressive strength, while still wet, is then determined at the end of the forty-eight hours period, in the manner specified in Section 6.

8. The freezing test shall be made as follows: The sample is immersed, as described in Section 7, for at least four hours, and then weighed. It is then placed in a freezing mixture or a refrigerator, or otherwise subjected to a temperature of less than 15° F. for at least twelve hours. It is then removed and placed in water, where it must remain for at least one hour, the temperature of which is at least 150° F. This operation is repeated ten (10) times, after which the sample is again weighed while still wet

from the last thawing. Its crushing strength should then be determined, as called for in Section 6.

9. The fire test is made as follows: Two samples are placed in a cold furnace in which the temperature is gradually raised to 1700° F. The test piece must be subjected to this temperature for at least thirty minutes. One of the samples is then plunged in cold water (about 50° to 60° F.) and the results noted. The second sample is permitted to cool gradually in air, and the result noted.

10. The following requirements must be met to secure an acceptance of the materials. The modulus of rupture for concrete blocks at twenty-eight days must average 150, and must not fall below 100 \* in any case. The ultimate compressive strength at twenty-eight days must average 1000 pounds per square inch, and must not fall below 700 in any case. The percentage of absorption (being the weight of water absorbed divided by the weight of the dry sample) must not average higher than 15 per cent, and must not exceed 22 per cent in any case. The reduction of compressive strength must not be more than 33½ per cent, except that when the lower figure is still above 1000 pounds per square inch, the loss in strength may be neglected. The freezing and thawing process must not cause a loss in weight greater than 10 per cent, nor a loss in strength of more than 33½ per cent, except that when the lower figure is still above 1000 pounds per square inch, the loss in strength may be neglected. The fire test must not cause the material to disintegrate.

**556. Cement Brick.**—In localities where a good coarse sand is found and cement is cheap, brick may be made of cement mortar which can successfully compete with clay building brick. One of the merits of cement brick is the true shape. Furthermore it is possible to make special shapes and sizes to exact dimensions almost as cheaply as the standard sizes.

For body brick an economy may be effected by adding coarse aggregate to the mix. The same considerations should obtain in determining proportions for cement brick as for concrete blocks. Mortar or concrete for brick making should have a strength of 1500 lb. per square inch at an age of one month when tested in the form of a prism or cylinder with a height equal to twice the diameter. Good cement brick one month old should have a modulus of rupture under transverse test of 400 lb. per square inch; half brick, at the same age bedded flatwise should have a compressive strength of 2000 lb. per square inch. Good cement brick should not absorb more than 6 per cent of water. The methods of testing used for concrete blocks can be applied to cement brick.

A noteworthy example of the successful use of cement brick in a large mill building at Plymouth, Mass., is cited in *Engineering News*, Vol. 61, p. 288. More than two million brick of 1 : 3 proportions were made at the site and used in the construction of the building. Four standard cement-brick machines each of which turned out about twenty brick every three minutes were used in molding. Brick on the outside of the building were faced with  $\frac{1}{8}$  in. layer of 1 : 2 mortar. The face brick absorbed about 1¼ per cent water and the body brick about twice that amount. The compressive strength of the body brick (whole brick placed flatwise),

\* The units are pounds per square inch.

increased from 2215 lb. per square inch at fifty-six days to 5160 lb. per square inch at nine months.

#### CEMENT DRAIN TILE AND SEWER PIPE

**557. The Advantages of Cement Pipe.**—Besides having the advantages common to other cement products cement pipe possesses two additional merits. They can be made true to form and of circular, oval or any desired cross-section. Furthermore, it is possible, by regulating the thickness of the shell or by reinforcing with steel, to make the pipe strong enough to withstand the pressure due to any superimposed ditch filling. Indeed reinforced pipe are being used for pressure conduits.\*

**558. Method of Manufacture.**—Portland cement of standard quality and a well-graded sand containing a large proportion of coarse particles are generally used for making the smaller sizes of tile or pipe. The proper proportions vary with the nature of the aggregate. For ordinary bank sands it is not safe to use a mix leaner than 1 : 3. With exceptionally good aggregate the proportions may be increased to 1 : 4. If the tile is to be laid in an alkali soil, only the best of sands should be used and the proportions should not be leaner than 1 : 3.† In making the larger sizes of drain tile and sewer pipe, it is good practice to employ concrete having a maximum size of coarse aggregate not over one-third of the thickness of the pipe wall. The mix should contain sufficient mortar so that a dense product can be secured. Proportions may vary from 1 : 2 : 3 to 1 : 2½ : 4, depending on the nature of the fine and coarse aggregates. The use of pit run gravel or run of crusher stone should not be permitted on account of the variation in the gradation of the aggregate.

In mixing, both machine and hand methods are used. Either is satisfactory when properly done (see Art. 489 and 490), but the machine mixing is far more economical. The consistency of the mix should be as wet as possible without interfering with the immediate removal of the molds. When the proper consistency is employed a web-like marking will be produced on the surfaces of the pipe. This is entirely lacking on products made from dry mixes.

In the larger plants the tile or pipe are filled and tamped by machines, of which there are two main types—the spiral-tamping machines and packer-head machines. In the former the mold is centered upon a horizontal table revolving about a vertical axis, and the mix is spouted into it. Tamping is accomplished by blows from a slender plunger which is moved up and down four or five hundred times a minute while the tile is being revolved. In some machines the plunger is held by a friction

\* The description of a long 36-in. pipe line under 70 ft. head may be found in *Eng. News*, Vol. 68, p. 248.

† See *Technologic Paper No. 44*, U. S. Bureau of Standards, entitled *Investigation of the Durability of Cement Drain Tile in Alkali Soils*.

grip so that its length can be automatically shortened as the mold is filled without changing the intensity of the blow.

With the packer-head machines the molds have no inner lining and are stationary. The packer-head revolves about a vertical central axis and also has an up-and-down motion. The head itself consists of a smooth cylindrical iron disc about 4 in. long and of the same diameter as the inside of the tile. It is surmounted by a set of adjustable radial vanes. In operating, the head is run to the bottom of the mold and the feeding spout opened. As the head is raised the material is forced outward against the mold by centrifugal force and the interior of the tile is finished by the troweling action of the outer portions of the vanes and the disc.

Tamping machines operate more slowly than the packer-head devices, but they produce a more uniform pipe. For these reasons cement drain tile are generally made by the packer-head method, while cement sewer pipe are made by tamping machines.

The method of curing outlined in Art. 497 should be followed if good results are to be secured. Nearly all plants now expose the tile to an atmosphere of wet steam for at least thirty-six hours, the temperature being kept above 70° F. The steam should be saturated so that condensation is in evidence on the walls of the curing chamber and on the product. Dry steam even at a high temperature will generally do more harm than good. After steam curing the pipe or tile should not be subjected to a temperature below 40° F. before they are two weeks old. To secure an impervious product it is necessary to sprinkle the pipe twice a day for at least one week after removal from the steam, especially if the weather is warm and the humidity low.

**559. The Testing of Cement Pipe.**—The rough field tests mentioned in Art. 287 may also be used to determine the quality of drain tile or sewer pipe. Many plants and most laboratories make absorption and crushing tests to ascertain quality. Both of these tests are made in the same manner as described in Arts. 291 and 293. Fragments of cement drain tile or sewer pipe should not absorb over 8 per cent of water, by weight. The table in Art. 324 shows the bearing strengths per lineal foot which the different sizes and classes of drain tile and sewer pipe should carry. A method for making freezing tests on drain tile appears in Art. 286.

Committee C-4, of the A. S. T. M. recommended in 1916 that sewer pipe should not show percolation under an internal water pressure of 10 lb. per square inch and should withstand an internal hydrostatic pressure sufficient to produce a tensile stress of 180 \* lb. per square inch in the pipe

\* Computed by the formula  $S_t = \frac{Rd}{2t}$ ; where  $S_t$  = unit tensile stress in pounds per square inch;  $R$  = internal pressure in pounds per square inch;  $d$  = internal diameter of pipe in inches; and  $t$  = thickness of shell in inches.



walls. The committee recommended that sewer pipe should be sufficiently tough to work with a chisel and hammer; also that the pipe should withstand without injury three blows from an 8-lb. cast-iron ball falling 18 in.

#### MISCELLANEOUS CONCRETE PRODUCTS

**560. Poles, posts and piles** are also cast or formed by centrifugal processes out of concrete. In most cases these forms must be reinforced to withstand flexure. The material is well adapted to use in posts and poles, but reinforced piles exposed to sea water are likely to suffer from corrosion of the reinforcement in the sections above high-water level. Reinforced concrete railroad ties have also been tried but without much success, the material being too brittle for such purposes.

**561. Other Forms.**—On account of the readiness with which concrete can be cast and modeled it forms an inexpensive material out of which the artist creates a great variety of handsome ornamental forms for the decoration of structures. Artistic urns, fountains, benches, and pieces of statuary molded out of concrete adorn many of our lawns, parks, and drives. Although the making of ornamental concrete is a comparatively new industry, yet the general characteristics of the material and the durability of the work thus far done, indicate an increasing demand for such products.

## CHAPTER XVI

### METALS AND THEIR ORES

**562. Metallurgy Defined.**—Metallurgy may be defined as the art of extracting metals from their ores, and their subsequent adaptation to the uses of man. As a division of engineering practice, the field is intermediate between those of the mining geologist and engineer, who exploit nature's deposits and take out the ores of value, and of the civil, electrical, and mechanical engineers, who are concerned with the fabrication of the metal into the finished structure or machine.

As implied in the definition, the field of metallurgy is broadly divisible into two parts: The recovering of the metal from the comparatively crude or complex forms in which it is taken from the ground; and the further manipulation or treatment of the metal, relatively pure or otherwise, to fit it to withstand best the varying conditions of service. To the user of materials the latter is in general the more interesting and important division of the art.

**563. The Metals of Construction.**—The metals used in considerable quantities in engineering construction are comparatively few in number: iron, copper, lead and zinc may be considered as the primary; and aluminum, tin, nickel, antimony, silicon, manganese, vanadium, chromium and tungsten, as the important metals of secondary rank. That is, the first four are usually the base metals in materials of construction, and are almost essential in engineering practice; the secondary metals are in general not so essential, or have their principal use because of their modifying influence when used in mixture with the metals of the primary division.

**564. The Utility of the Metals in Construction.**—The utility of any metal depends upon its adaptability and workability and upon its cost. The two first named are functions of the physical properties of the material; the cost depends upon the abundance and accessibility of the metal, its form of occurrence, and the ease of extraction and treatment. The properties of importance in metals for construction purposes are: tenacity, elasticity, ductility, malleability, hardness, specific gravity, fusibility, viscosity, color, corrodibility, expansibility, conductivity (electrical and thermal) magnetic quality, electrical potential, and frictional quality.

*Iron* is by far the most important of the metals used in engineering construction. It is very abundant in nature, which fact, together with

The following are the most important chemical combinations in which the more common metals are found:

Oxides or hydrated oxides—	iron, copper.
Sulphides—	copper, lead, zinc.
Carbonates—	lead, zinc, iron, copper.
Native—	copper (silver, gold).
Silicates—	zinc.
Chlorides—	lead.

**566. Economic Value of Deposits.**—Whether or not a metalliferous deposit is an ore depends upon economic considerations. The commercial feasibility of recovery depends upon:

(1) Nature of the metal—the more valuable the metal to be recovered, the greater the extremes to which one may go in such recovery, and still show a profit in the operation.

(2) Richness of the deposit—the greater the percentage of metal present in the deposit, the more feasible is the prospect of extraction; other things equal. A deposit with 50 per cent of iron content is high grade, while one with 30 per cent is lean. Copper ores, on the other hand, vary from 5 to 1 per cent in the largest workings of this country. The average amount of gold recovered from straight ores in the United States in 1914 was valued at \$6.95 per ton, indicating a yield of approximately  $\frac{1}{3}$  oz. per ton or 0.001 per cent.

(3) Association—two or more metals may be associated in an ore, and recovery of both will show a net profit, whereas if the separate metals were alone in their individual proportions extraction would not be feasible. Very great values in silver are recovered in lead extraction from certain ores, although the percentage content is so low in many cases as to be apparently negligible. Most of the nickel is obtained from ores where it is associated with sufficient copper to warrant recovery of both metals. Certain iron ores have an added value conferred by the association of nickel or manganese. Again the association may not be of a metallic nature, but may, even as part of the gangue material, save additions in the extraction process, and in consequence warrant the treatment of otherwise too lean ores. Thus the lime associated with some European and Southern ores enhances their value. On the other hand, foreign substances of a detrimental nature may be present in the deposit, and either destroy the possibility of economical recovery or so increase the costs of the operation as to lower the grade of an otherwise rich ore. High phosphorus in iron ores, or the high titanium content of many of the iron deposits of New York, are instances in point.

(4) Situation—Geographical location with respect to facilities for transportation, and accessibility to smelting centers and markets for their

products are factors for consideration. Immense deposits of rich iron ores in Brazil are practically untouched at the present time because of transportation difficulties and costs. Again, location of the deposit with respect to ground surfaces, as determining the methods and cost of mining, is an item in the determination of its value. Finally, extent of the deposit, both as to total quantity and character of distribution, is an important factor. To warrant exploitation, an iron deposit must have large tonnages in sight in a bedded form of considerable thickness. The more valuable the metal, the more feasible it becomes to take out the mineral occurring in veins, pockets, or streaks.

(5) State of the art—cheaper methods of extraction may be developed which will show a profit in the recovery of metals from ores of too lean a character to be treated by present-day methods; or such processes may eliminate some element now detrimental. Also as the high-grade ores become exhausted it becomes necessary to draw upon sources of supply of less rich character, or at more remote or inaccessible places.

**567. Preparation of Ores for Extraction of Metals.**—After the ore has been mined, it is usually necessary to prepare it for the extraction process, by some method of ore dressing or concentration. The object is to get rid of some harmful constituent or some of the gangue material by a method depending upon different physical or chemical properties than those of the extraction operation, and accomplishing the result in a more effective and economical manner. Naturally the greater the intrinsic value of the metal to be recovered, the greater is the amount of ore dressing which can be employed, other things being equal. On account of the fact that it is usually more feasible to extract the metal at a place favorably situated with respect to fuel, etc., and remote from the mine mouth, elimination of a considerable portion of the gangue at the mine will minimize transportation of such useless material, and such saving will often more than repay the cost of concentration aside from the advantages resulting from a material more favorable for extraction.

Concentration methods usually begin with a crushing and sizing of the ore to a point as close as economical considerations will warrant, to the theoretical limit of mechanical disintegration of the useful mineral from the gangue. Varying physical or chemical characteristics of the two are the basis for the separation. Washing with water will carry away clays or substances which will form an emulsion or go into solution. Specific gravity differences will enable one to effect a separation by flotation in water, oil, etc., usually by some means of controlling its velocity of flow or pulsation. Magnetic separation is often of value, particularly when iron minerals are present.

**568. Principles of Extraction of Metals.**—Metallurgical extraction methods are divisible into two groups: Wet methods involve solution of

the metal sought, and recovery by precipitation; while useful in some fields, particularly gold and silver, they are relatively unimportant in the metallurgy of the metals used in engineering construction. Dry methods predominate; usually called smelting, and effected with the aid of heat. After the ores have been properly prepared for treatment, the smelting operation consists essentially of a reduction of the metal from its chemical combination in the mineral, a fusion of the metal and the gangue as slag, with recovery of the metal. The result may be obtained in one operation and furnace; or it may require several to properly fit the metal for use in the arts.

The chemical reactions involved in the recovery of the metals in smelting are mainly oxidation with air, metallic oxides, and carbon dioxide; and reduction by means of carbon, carbon monoxide, hydrogen, sulphur, or certain metals.

Heat is often employed preliminary to smelting, to effect a change in the chemical nature of the mineral or ore, without fusion or reduction to the metallic form. Drying, calcination, or roasting involve the driving off of the moisture, the dissociation of hydrates or carbonates with elimination of water or carbon dioxide, and the removal of sulphur by oxidation and volatilization as sulphur dioxide.

## CHAPTER XVII

### REDUCTION OF IRON FROM ITS ORES

**569. The Economic Importance of Iron and Steel.**—The iron and steel industry has had its principal development in the United States and Europe. Production of pig iron in recent years is shown in Table 1.

**TABLE 1.—QUANTITY AND VALUE OF PIG IRON PRODUCED IN LEADING COUNTRIES OF THE WORLD.**

Country.	1905		1910		1915	
	Long Tons.	Dollars.	Long Tons.	Dollars.	Long Tons.	Dollars.
United States.....	22,992,380	382,450,000	26,674,123	412,162,486	30,384,486	401,409,604
Germany.....	10,814,018	.....	14,559,590	.....	11,603,914	.....
Great Britain.....	9,592,737	.....	10,216,745	.....	8,793,659	.....
France.....	3,028,383	.....	3,968,746	.....	4,674,950	.....

In addition, Belgium, Russia, Austria, and Norway and Sweden have appreciable production. By far the greater proportion of pig iron produced in the United States is made from domestic ores, although there is a considerable tonnage of ore imported from Cuba, Canada, Spain, Sweden, Africa and Chile to supply seaboard furnaces. Germany is largely dependent upon iron ores obtained from beyond her borders. Her chief supplies are from the Lorraine fields, extending into Luxembourg, Belgium, France and Germany. Great Britain has important deposits of iron ore on her northeast and northwest coasts, but is depending more and more upon importation from foreign fields. About 30 per cent of her supply in 1913 was imported, principally from Spain, Algeria, Norway and Sweden.

**570. The Native Sources of Iron Ores.**—Iron ores are mined in many states of this country. There are three great producing districts: Lake Superior, centering about the head of that lake in Wisconsin, Michigan and Minnesota; Southern, chiefly in Alabama, near Birmingham; and New York, mainly in the Adirondacks near Lake Champlain. Some tonnage is obtained in Ohio, New Jersey, Pennsylvania, Virginia, Georgia, Tennessee, and Colorado. The Lake Superior district produces about four-fifths of the annual tonnage of this country. Southern ores are smelted largely in the vicinity of the mines. New York ores find outlet locally and in eastern Pennsylvania furnaces; while almost all of the Lake Superior ore

is shipped from 600 to 1000 miles to smelting centers. The greatest proportion is sent to lower Lake Erie ports for transshipment by rail to furnaces in the Pittsburgh districts; while a considerable tonnage is smelted in the region centering about Chicago. The Pittsburgh district has long held the supremacy as the steel-making center of the United States, largely because of the economic advantages of good fuels (coal, coke, natural gas and oil) and accessibility to the consuming markets. With the trend of population westward, and the changed conditions due to the development of modern coking processes, it is likely that Chicago will become an increasingly greater center of activity. Production in Colorado or far western districts will be entirely to satisfy local demands.

**571. Classes of Iron Ores and Their Characteristics.**—The ores of iron are classed according to the iron mineral which is predominant. They are, in the order of theoretical percentage of iron content in the mineral:

*Magnetite*;  $\text{Fe}_3\text{O}_4$ —72.4 per cent iron. It is steel gray to black in color; very hard and strongly magnetic. Magnetite is not a very prominent ore in the United States, but is mined to a considerable extent in New York and New Jersey. There are large tonnages in New York which are not exploited at the present time because of associated titanium, which causes difficulty in smelting. Magnetite is the important ore of Sweden, where it is of high purity.

*Hematite*;  $\text{Fe}_2\text{O}_3$ —70 per cent iron—is red to brown in color, and occurs in some deposits in a hard, lumpy condition, whereas in others it is soft and of fine texture. Hematites form the principal ores of the United States; the ore of Lake Superior and much of that of the South is of this character. Hematites are also important in the west of England, Spain, Cuba, Chile and Africa. Immense deposits of high grade ore exist in Brazil, but have not been exploited to any extent as yet because of transportation difficulties.

*Limonite*;  $2\text{Fe}_2\text{O}_3 + 3\text{H}_2\text{O}$ —59.9 per cent iron, 14.5 per cent water of combination. This is a hydrated hematite, is soft and of a yellow color. It becomes red hematite by dehydration at high temperature. By partial dehydration, ores of brown color, called brown hematites, result. These are prominent in the southern deposits of the United States, and in the Minette district on the German-French border. Limonites are important in the Cuban deposits.

*Siderite*;  $\text{FeCO}_3$ —48.3 per cent iron, 41.4 per cent  $\text{CO}_2$ —also called spathic ore. The principal deposits are in Germany and Austria, in Wales and the northwest district of England. Normally the ore is hard and stonelike with a gray color, but is often associated with clay or carbonaceous matter, and in the latter case the color is black or streaked with black. Carbonate ores were formerly important in Pennsylvania.

While iron silicates and sulphides are plentiful in nature, they are of no importance at the present time as ores.

Iron ores occur in all conditions in nature, from hard ores requiring blasting, to soft ores which can be mined directly with steam shovels. Successfully mined deposits are often thousands of feet below the surface of the ground, and again may be so close to the surface as to be capable of open cut mining after a variable amount of stripping of top soil.

The relative production of the various iron ores in the United States during recent years is given in Table 2.

TABLE 2.—RECENT PRODUCTION OF IRON ORE IN THE UNITED STATES

Kind of Ore.	1905.		1910.		1915.	
	Long Tons.	Per Cent.	Long Tons.	Per Cent.	Long Tons.	Per Cent.
Hematite*.....	37,567,055	88.3	51,367,007	90.1	52,227,324	94.1
Brown Ore†.....	2,546,662	6.0	2,993,744	5.2	1,488,709	2.7
Magnetite.....	2,390,417	5.6	2,631,835	4.6	1,807,002	3.2
Carbonate.....	21,999	0.05	22,320	0.04	3,455	0.006
Total.....	42,526,133	100.00	57 014,906	100.00	55,526,490	100.00

\* Anhydrous sesquioxide.

† Hydrated sesquioxides.

**572. Elements Associated with Iron Ores.**—Typical analyses of iron ores are given in Table 3.

As will be noted in the table, the iron content of different ores varies from 30 per cent to upwards of 50 or even 60 per cent. The Lake Superior ores are very rich, while those of the South are comparatively lean. Many ores, and this is true of certain of the southern deposits, have the disadvantage of the lean iron content offset to a degree by the association of lime, alumina and silica in proper proportions to be self fluxing, thereby forming suitable slags without purchase of the necessary flux (usually limestone) to ensure fluidity. The gangue is earthy material, of which the principal constituents are usually silica and alumina. Increasing amounts of such gangue lower the value of the ore by decrease of iron content and corresponding increase of handling, fuel, and flux costs per ton of pig iron. Alkalies or basic constituents may have the offsetting advantage of decreasing the amount of flux required in smelting. Sulphur and phosphorus are always present in varying degree in iron ores. Up to 1 per cent of the former may be successfully removed to the required limits in blast furnace smelting, but requires increased fuel and flux; thus increasing sulphur lowers the value of the ore. Phosphorus, on the other hand, is eliminated only by special processes in the refining of the crude blast furnace product to steel; the necessity of such selection of a more



TABLE 3.—CHEMICAL ANALYSES OF VARIOUS IRON ORES

(From *Tenth Census Reports*, Vol. 15)

Components.	KIND OF ORE.						
	Hematite.			Limonite		Magnetite.	Carbonate.
	25.	538.	252.	267.	967.	738.	995.
Fe.....	62.65	59.52	50.04	53.60	44.60	48.91	43.05
P.....	0.011	0.005	0.108	0.641	0.365	0.015	0.115
S.....	0.018	0.020	0.179	0.085	0.011	0.548	0.045
FeO.....	1.81	0.78	0.22	0.46	.....	21.99	54.385
Fe <sub>2</sub> O <sub>3</sub> .....	87.47	84.16	71.15	75.99	63.714	44.75	1.071
SiO <sub>2</sub> .....	4.42	12.17	11.23	10.52	.....	12.18	.....
Al <sub>2</sub> O <sub>3</sub> .....	2.37	2.21	4.99	5.58	1.090	0.79	1.457
MnO.....	0.25	.....	.....	0.23	.....	0.10	3.254
Mn <sub>2</sub> O <sub>4</sub> .....	.....	.....	.....	.....	0.455	.....	.....
CaO.....	0.26	0.04	0.25	2.23	0.180	3.10	0.540
MgO.....	1.58	0.04	0.25	0.52	0.324	11.72	0.540
FeS <sub>2</sub> .....	0.034	.....	0.337	0.124	.....	1.031	.....
BaO.....	.....	0.10	.....	.....	.....	.....	.....
CoO.....	.....	.....	.....	.....	.....	.....	0.010
Co <sub>2</sub> O <sub>3</sub> .....	.....	.....	.....	.....	0.040	.....	.....
SO <sub>2</sub> .....	.....	0.05	.....	0.05	0.027	.....	0.112
K <sub>2</sub> O.....	0.38	0.15	0.14	0.17	.....	.....	.....
Na <sub>2</sub> O.....	.....	0.07	0.11	0.01	.....	.....	.....
P <sub>2</sub> O <sub>5</sub> .....	0.26	0.011	0.246	1.469	0.836	0.34	0.263
CO <sub>2</sub> .....	0.12	0.04	0.13	0.09	.....	0.40	35.340
C.....	0.01	.....	0.25	0.04	.....	.....	.....
H <sub>2</sub> O (Hygr.).....	0.19	0.12	0.42	1.02	} 11.980	0.29	{ 0.923 by dif.
H <sub>2</sub> O (com.).....	1.12	0.37	10.20	1.69		3.79	
Insol. res.....	.....	.....	.....	.....	21.940	.....	2.105
Total.....	100.40	100.311	99.923	100.193	100.586	100.175	100.000
Sp. gr.....	4.534	.....	3.706	4.110	.....	.....	.....
Locality.....	Chapin	Pilot Knob	Tecumseh	Attala	Hellertown	Tilly Foster	Hellertown
State.....	Mich.	Mo.	Ala.	Ala.	Pa.	N. Y.	Pa.

costly method means that phosphorus content percentage greater than 0.05 per cent in an ore carrying 50 per cent of iron, has a marked effect in lowering the value. Phosphorus of 0.001 per unit of iron is the limiting content dividing iron ores into the two great divisions of Bessemer and non-Bessemer. All good steel must not exceed 0.10 per cent of phosphorus; the elimination of any excess costs money, and the presence of such excess in the ore lowers the grade.

**573. Preliminary Treatments for Iron Ores.**—Iron ores usually undergo no treatment preliminary to smelting; except carbonates, which are calcined to drive off the carbon dioxide. Sulphur in excess of that suitable for economical smelting can be lowered by roasting and volatilization as sulphur dioxide. Much of the magnetite mined in New York is crushed and treated by magnetic separation, whereby a considerable proportion of the non-magnetic gangue and phosphate constituents are removed, and two-fold enrichment in grade is the result. Washing of the brown ores of the South is practiced to remove excessive clay, and this method is coming into increasing importance to remove sand and clay from certain of the Lake Superior ores. Drying of ores is necessary in many cases, and is attracting more and more attention; elimination of excessive moisture results in increased smelting economies, and in addition the cost of drying may be more than compensated for by the saving in freight charges on the water content. Agglomeration of fine ores by briquetting with binders of slag, tar, etc.; or nodulizing or sintering by partial fusion, are being successfully practiced. At present the employment of these methods is chiefly in the utilization of flue dusts formed in blast furnace operation.

**574. Fundamental Principles of Extraction of Iron.**—The fundamental chemical principles in the extraction of iron from the ores are very simple. Heating the ores in the presence of a reducing agent (usually carbon or carbon monoxide) will result in the formation of carbon monoxide or carbon dioxide, liberated as a gas, and metallic iron. Reduction of red oxide of iron,  $\text{Fe}_2\text{O}_3$ , by carbon monoxide will begin at temperature as low as  $200^\circ\text{C}$ ., and proceed by progressive formation of  $\text{Fe}_3\text{O}_4$ ,  $\text{FeO}$  and  $\text{Fe}$ . Reduction to metallic iron is practically complete at temperature as low as  $800^\circ\text{C}$ . very much below the fusion point of iron, which melts at  $1505^\circ\text{C}$ . when pure, and may have its fusion point lowered to  $1150$  or  $1200^\circ\text{C}$ . by the presence of associated elements or impurities.

**575. Ancient Methods of Extraction.**—This ready reducibility of iron oxide to metallic iron at moderate temperatures was the basis of the ancient methods of manufacture with apparatus of the simple hearth or forge type, resembling a crude blacksmith's forge, and with such forced draught only as could be obtained from bellows, in many cases hand-operated. A small charge of ore together with the necessary charcoal for fuel and reduction were heated for a sufficient time to effect reduction of the iron to a pasty mass. Ores of a minimum of associated gangue material were selected, much of which could be eliminated by combination with iron oxide to form a slag of fairly low fusion point, but necessarily with considerable loss of iron in the slag. The resultant mass of spongy iron with its entrained slag, was taken from the forge and hammered to a rough bloom or bar. Much of the slag was thus eliminated by pressure; but, just as it is impossible to eliminate all of the moisture from a sponge

by squeezing, so it was impossible to squeeze out all of the slag. The product of the forge was a malleable material with varying amounts of occluded slag which was extended by forging into long filaments; a product identical in physical structure with the wrought iron of more modern times.

**576. Direct and Indirect Methods of Producing Ductile Ferrous Metals.**—Since for the larger part of the iron which is used in the arts, some degree of its inherent malleability and ductility are desired, any process of manufacture whereby the product is obtained in this condition in a single operation represents the ideal. The "direct process" still makes its appeal to the inventor; actually, however, the amount of product so produced is negligible in a consideration of the annual tonnage. Unfortunately, such processes have many disadvantages; intermittent character, inefficient types of furnace, high labor costs, and large losses of iron in the slag. And especially, because detrimental impurities such as sulphur and phosphorus are only removed, or most economically so, in processes which bring the products to fusion, and where slags of special composition and volume can be formed. "Direct" methods have given way to-day to the so-called "indirect," whereby the malleable end product of greatest use in the arts is the result of a two-step operation; a preliminary smelting producing a relatively crude pig iron, and subsequent refining or conversion of this into steel of the desired composition and properties for various uses.

The most efficient furnace is a shaft type where the fuel and charge to be treated are in direct contact; where these solid materials can be charged at the top and descend by gravity to the base of the stack; where in this descent they may meet a counter current of ascending gases, formed from the combustion of the fuel near the bottom of the shaft; and, finally, where uninterrupted operation is made possible by fusion of all of the products and their withdrawal from the furnace at will in the molten condition. By attainment of suitable temperatures and character of slags, the latter may be easily separated from the iron in the molten condition, due to the different densities and lack of mutual solubility.

**577. The Development of the Blast Furnace.**—The modern blast furnace, which embodies all of the above characteristics, is the result of evolution and development. The first shaft type furnaces were designed primarily for more efficient and economical operation; the product was malleable material, removed intermittently in the unfused condition. With higher temperatures resulting from increased height of stack and better blast, the spongy mass of iron absorbed carbon from the fuel, and its fusion temperature was lowered from 1500° C. to between 1100° and 1200°. This was below the working temperature of the furnace, and the result was a molten product; but the absorption of carbon up to the limit

of saturation of the temperature changed the product from a malleable material to a brittle and relatively weak cast iron, while at the same time there was elimination of slag because of liquation in the fused condition. The era of this development was about 1400 to 1500 A.D. The limited demand for a malleable product, however, practically nullified the advantages of the method until the birth of the puddling process in 1784 gave a refining on a scale commensurate with the capacity of the shaft type of furnace.

**578. Description of a Modern Blast Furnace.**—In the early part of the eighteenth century, the blast furnace for the manufacture of pig iron was a square masonry stack, with four tuyeres near the base for the introduction of cold blast furnished by a crude blowing equipment. Its dimensions were about 10 ft. across by 30 ft. high, and production was from 30 to 40 tons of pig iron per week with a consumption of 3 tons of fuel per ton of iron. The furnace was usually placed at the side of a hill, so that materials of the charge could be hauled by team to the level of the top.

Important events in the development of the blast furnace have been the substitution of coke for charcoal at the beginning of the eighteenth century, and the invention of the steam engine in the latter part of that century; both of these made possible larger furnaces, more rapid driving and less restriction in location of plant. Hot blast was introduced in the early part of the eighteenth century, and with the resulting increase of temperature of the hearth, there was an increase in furnace production per unit of capacity, with decrease of fuel consumption. Closing of the top of the furnace enabled the top gases to be collected and utilized for blast heating, boiler firing and the like.

Developments in late years have been chiefly of a mechanical nature—enlargements of capacity, the adoption of the circular steel shell with brick lining, evolution of furnace lines as dictated by experience and by the changing character of the available ores; and most especially, in the installation of mechanical handling appliances to eliminate hand labor to the greatest degree, and make for larger output and more economical operation.

The modern blast furnace has a daily capacity from 100 to 700 tons, 500 tons may be considered fairly standard. Fuel consumption is approximately one ton of coke per ton of iron—it varies from 1700 to 3500 lb. according to character of ore and details of practice. Furnace diameters are from 15 to 25 ft. at the base and the height is from 60 to 90 ft.

A section of a modern blast furnace is given in Fig. 1. It is of the vertical shaft type, of two cone frustums, with their bases together, and resting upon a short cylindrical lower section. The long, gradually outward flaring-portion furnishes the shaft for the descent of the charge, for the absorption of heat from the ascending current of gases,

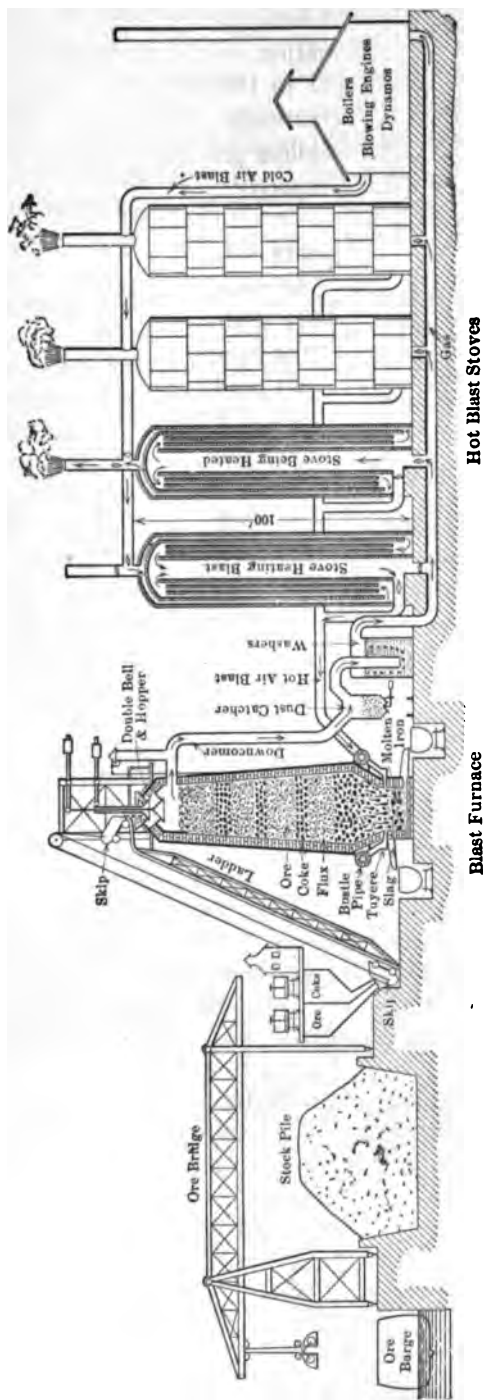


FIG. 1.—Diagrammatic Sketch Showing Scheme of Operating a Blast Furnace.

and for the step by step reduction of the ore to metallic iron. This operation begins shortly below the top, and is practically completed by the time the charge reaches the part of the furnace of greatest diameter. The outward flare is to take care of the increasing volume of material due to any swelling of the charge, and to allow of easy descent without danger of sticking. The lower frustum, called the "bosh" of the furnace, is the hottest part and forms the zone of fusion. There is a rather sharp flare of the walls inwards in the descent in accordance with the diminished volume of material in this zone. Also this bosh angle tends to promote arching, thus supporting the charge column, and to contract the base of the furnace to the diameter needed for the crucible and for penetration of the blast. The lower cylindrical portion is the crucible, or hearth, and serves as a reservoir for collection of the iron and slag between the periodic taps.

The upper shaft of the furnace is of steel-plate construction and sets upon a "mantle ring" in turn supported by 8 to 12 columns. This portion is

lined with fire-brick to a thickness of 4 to 5 ft. The bosh zone in general is of brick work from 2 to 3 ft. thick, and without a metal shell. Support is furnished by a series of heavy steel bands, and several rows of "bosh plates" through which water is circulated are set in the brick work to assist it in withstanding the high temperature of the fusion zone. The crucible is surrounded by a heavy jacket of segmental steel castings, and has a fire-brick lining about 3 ft. thick.

The blast is introduced through 8 to 12 tuyeres, or nozzles, made of bronze or copper, and water cooled; these are connected by suitable pipes to the large blast main surrounding the furnace and connected with the stoves and blowing engine. At the lower level of the crucible is the tap hole, from which the pig iron is run into sand beds or large ladles. Just below the tuyeres is a "cinder notch" through which the slag accumulated between taps of iron can be flushed. Near the top of the furnace are one or two openings for the escape of the gases. These gases are led by a pipe to the stoves, boiler, and other points of consumption. The top of the stack is closed by a conical bell and hopper; in the most modern furnaces this is double, to prevent escape of gas which would occur during the lowering of a single bell in charging. Some furnaces are charged by hand buggies taken to the top by an elevator. The best construction is an inclined bridge up which skips are hauled and automatically dumped after being filled at pits at the base of the furnace.

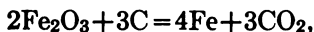
Typical dimensions of a modern blast furnace of 400 tons daily capacity are—crucible, diameter 14 ft., bosh, 21 ft., top, 14 ft., height, 80 ft. Such a furnace would require daily about 1500 tons of charge material—ore 800 tons, coke 500 tons, flux 200 tons. This material is charged at the top in alternate layers as required to keep the furnace full. The molten iron is tapped off every six hours and cast into pigs of 150 lb. weight or taken away in ladle cars for conversion into steel. The slag, which may be of greater total weight than the iron, floats on top of the iron, and is flushed off at intervals and usually dumped as waste material either molten or after water granulation. Some of the slag finds outlet in cement making, for road ballast, and for various other uses.

**579. Accessories to the Blast Furnace.**—The top gases are combustible. They contain carbon monoxide diluted with much carbon dioxide from the reduction of the ore and decomposition of the limestone, and large volumes of nitrogen derived from the air of the blast. This gas is freed from dust to varying degrees by dust catchers, scrubbers and washers, and used in part for heating the blast; the rest is available for power purposes. The air blown into the furnace is preheated to about 500 or 600° C. by four stoves, Fig. 1. For the 400-ton furnace noted above each stove would be about 20 ft. in diameter and 100 ft. high, consisting of a steel shell lined with fire-brick and containing a combustion chamber and a heat-absorbing

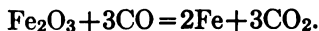
chamber of fire-brick checker work. The stoves operate on the regenerative principle—combustion of the gas for three hours results in sufficient heat absorption to enable each stove to heat the entire cold blast for one hour. Thus with four stoves per furnace, changes are made each hour, and three stoves are always on gas and one on air.

Besides the accessories used in heating and cleaning the blast, power is needed for ore handling and for pumping the enormous quantities of water used for cooling. Also 2500 to 3000 horse-power is required for supplying the 35,000 cu. ft. of air required per minute for a 400-ton furnace. Blast pressure is about 16 lb. per square inch.

**580. The Essential Reactions in Extracting Pig Iron.**—The underlying chemical principle in the extraction of iron from its ores is comparatively simple. The ores are oxides in the natural state, or converted to oxides by preliminary roasting or calcination. The problem involves reduction of an oxide (ore) by a reducing agent (C) with the aid of heat (combustion of C). The ideal reaction would be



requiring 36 units of carbon to yield 224 units of iron, or a ratio of 6 iron to 1 reducing agent. In addition to the above, there would be needed the heat required for continuance of the above reaction, since that furnished by the combustion of carbon to carbon dioxide is only  $\frac{1}{3}$  of the quantity necessary to dissociate the oxygen from the union with the iron. And if the resultant iron is to be fused, together with the associated gangue of the ore, still more fuel would be required. Actually the conditions in the blast furnace result in variation from the reaction cited above. The ore and coke are charged together at the top of the furnace, and descend slowly through the shaft in contact. No combustion of the fuel takes place until it reaches the air supply at the tuyere zone, where intense heat generation results. The products of combustion pass rapidly up the stack, giving up heat to the charge in their ascent and causing a temperature gradient in the furnace from 1600° C. in the tuyere zone to 200–300° C. at the top of the furnace: The carbon dioxide formed at the tuyeres undergoes almost immediate change to carbon monoxide in the presence of the large coke bed, and as such passes upwards through the stack. As a gas, it has great penetrating power; furthermore it will begin to reduce iron oxide at temperatures as low as those of the blast furnace top, and will practically complete the reaction at temperatures at which solid carbon begins to act. Consequently, although the ore and carbon are in contact, with ores of reasonably porous texture, the reduction may be considered as effected through the agency of the carbon monoxide. The ideal reaction is:



Under these conditions there would result 112 units of iron for each 36 units of carbon, or a ratio of 3 to 1. Unfortunately, conditions of chemical equilibrium govern the approach to the ideal which can be obtained, and experience indicates that with the temperatures, pressures and materials prevailing in the blast furnace, the ratio of carbon monoxide to carbon dioxide is usually about 2 to 1. The blast furnace reduction of iron may be typified approximately as follows:



The result is a production of 112 units of iron with the consumption of 108 units of carbon, or an approximate ratio of 1 to 1. The combustion of the 108 units of carbon to CO in the tuyere zone results in sufficient heat generation to fuse the iron and slag, and in the surplus heat needed to complete the reactions in the upper zones of the furnace. Passing from the shaft is a gas which has considerable combustible value because of the high carbon monoxide ratio, but which is diluted by the carbon dioxide and particularly by the large volumes of inert nitrogen accompanying the oxygen in the air blast.

The simple equation of reduction given above is complicated by the successive stages taking place in the zones of descent, by the charges accompanying varying physical conditions in charge and atmosphere, and by the reversibility of reactions and conditions of chemical equilibrium as influenced by mass relations.

In actual blast furnace practice fuel ratios are subject to considerable variation with varying richness of ore and coke and details of furnace construction and operation. From 1600 to 2600 lb. of coke and from 700 to 1200 lb. of limestone per gross ton of pig iron is about the range for modern furnaces working on Lake Superior ores. In 1915 the averages in pounds per gross ton of pig iron for constituents of the charge in the blast furnaces of the United States was as follows:

Coke.	Limestone.	Ore.	Cinder, etc.
2252	1107	4128	334

In Fig. 2 is given a diagrammatic representation of the typical reactions of the blast furnace, together with the weights of the several materials making up the charge and the products. These are based upon a dry ore containing 60 per cent iron, coke containing 87 per cent carbon, pure limestone, pig iron with 4 per cent carbon and 1 per cent silicon, and a tunnel head gas of 1 part CO<sub>2</sub> to 2 parts CO, by volume. Beginning at the upper left corner of the diagram and following the sequence of figures



to the left, one observes how the ore, coke and limestone are affected as they gradually move downward in the furnace, also how the gases are transformed as they pass upward through the charge.

**581. The Reduction of Impurities in Iron Ores.**—Besides reducing iron ore to the metallic state, the blast furnace causes a variable reduction of the constituents of the gangue, flux, or fuel, and an absorption of the liberated metalloids by the iron, to its benefit or detriment. By regulation of temperature and slag characteristics, the blast furnace manager has a

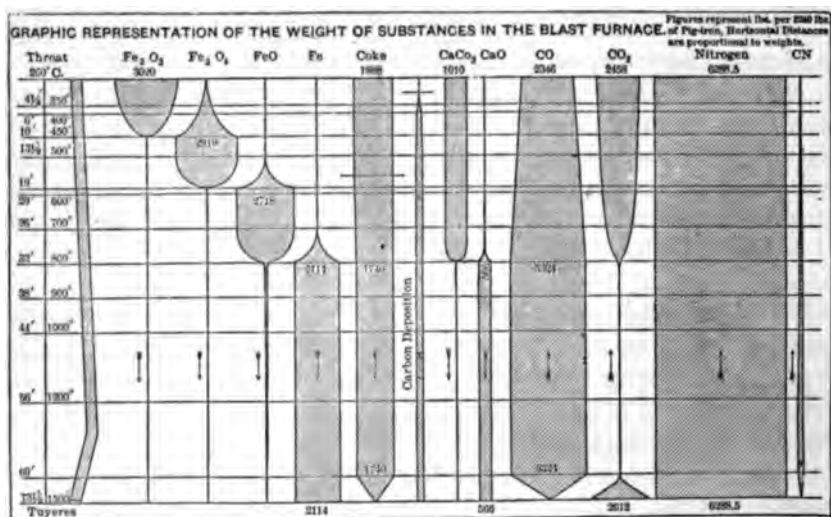


FIG. 2.—Showing Changes which Occur in a Blast Furnace Charge. (From Campbell's *Manufacture and Properties of Iron and Steel*.)

considerable degree of control over certain of these. Carbon is taken up from the fuel in the fusion zone in fairly fixed amounts of  $3\frac{1}{2}$  to 4 per cent. The percentage varies with the amounts of other elements in the iron; silicon decreases total carbon; manganese increases it. Whatever phosphorus is present in ore or fuel is readily reduced to the elemental form and absorbed by the iron. It cannot be controlled under the reducing conditions of the blast furnace, and appears in the iron to the full extent which it may be present in the charge. The presence of phosphorus is detrimental in steel-making irons, and in cast irons, except where considerations of fluidity overtop those of strength.

Manganese is present to varying extent in the ores of iron. Part of it is reduced and appears in the iron, the amount varying with the quantity in the ore and running conditions in the furnace. For ordinary grades of pig iron, control of furnace conditions is not regulated by manganese

consideration; the relative amounts in iron and slag are the result of conditions determined by elements more vitally affecting quality.

In ordinary grades of pig iron silicon is desired in fairly fixed percentages, varying from 0.75 to 3.5 per cent according to use. With increasing temperature in the hearth zone, there is increasing tendency for reduction of  $\text{SiO}_2$  by carbon; offsetting this is the strong affinity of lime or other bases to unite with  $\text{SiO}_2$  to form silicates. Such silicon as is set free in the metallic state is absorbed by the iron; unreduced  $\text{SiO}_2$  forms part of the slag. High silicon in the pig iron results from high temperatures of running (usually the accompaniment of higher fuel ratios) coupled with slags of high basicity; that is with the lime or other strongly basic flux held to a maximum.

Sulphur is an undesirable element in all iron and steel, in amounts exceeding about 0.10 per cent. And since the amount present in ores of a blast furnace charge may approach 1 per cent, and since the removal of sulphur is best effected under reducing conditions, its elimination by proper control of blast furnace conditions is a most important function of operation. As iron sulphide it is absorbed by the iron; as calcium sulphide it will be dissolved by proper slags; elimination from the pig iron depends largely upon the formation of calcium sulphide. Excess of lime above the amount needed for complete satisfaction of the silica (neutralization of acid by base) together with high temperature, favor the reduction of lime ( $\text{CaO}$ ), in which case the free calcium will take hold of any sulphur to form calcium sulphide, which is soluble to limited extent in the hot, limey slag. Low sulphur pig iron is the result of high temperature furnace operation with high-lime (basic) slags of sufficient quantity to ensure solution of all of the calcium sulphide formed before the saturation limit is reached.

**582. Grades of Pig Iron.**—Analyses of typical grades of pig iron are given in the following table. It will be noted that the carbon content is not given. Although it is the most important element in cast irons, its percentage is fairly constant—3.5 to 4 per cent—in pig iron, and is largely governed by the amounts of other elements, as noted heretofore.

Foundry irons are intended for the manufacture of iron castings. Softness and soundness are desired with reasonable strength. Increasing the silicon content promotes the formation of graphitic carbon, and thus tends to increase softness. Sulphur promotes formation of combined carbon, and tends to increase hardness. High phosphorus decreases strength but increases fluidity and therefore is of value in the manufacture of thin castings, stove-plate, etc. Foundry pig irons are of higher grade (and value) with increasing silicon and decreasing sulphur content. Gray forge iron is a lower or off grade of foundry pig, often used in puddling.

Malleable Bessemer pig iron is intended for the production of malleable cast iron. Silicon is held within restricted limits to ensure a hard white casting which will be subsequently converted to the proper soft gray tex-

	Si.	S.	P.	Mn.
No. 1 Foundry.....	2-3	<0.035	0.5-1.0	<1.0
No. 2 Foundry.....	2-2.5	<0.045	0.5-1.0	<1.0
No. 3 Foundry.....	1.5-2	<0.055	0.5-1.0	<1.0
Gray Forge.....	<1.5	<0.100	<1.0	<1.0
Malleable Bessemer.....	0.75-1.5	<0.050	<0.20	<1.0
Bessemer.....	1-2	<0.05	<0.10	<1.0
Basic.....	<1	<0.05	<1.0	<1.0
Basic Bessemer.....	<1	<0.05	2-3	1-2

< = below or less than

ture by annealing. Phosphorus is kept low because of strength considerations, but not too low to destroy fluidity in casting.

Bessemer pig iron has a silicon content sufficiently high for the fuel requirement of the acid Bessemer steel-making process. Phosphorus must be below the limiting amount for a satisfactory steel.

Basic pig irons are intended for steel making by basic processes in which phosphorus can be removed. Economic slag conditions necessitate low silicon. In the basic Bessemer method high phosphorus is needed for its fuel value; it is not essential in the basic open-hearth method.

**583. Slags.**—Blast-furnace slags will in general have a silica plus alumina ( $\text{SiO}_2 + \text{Al}_2\text{O}_3$ ) content about equal to the lime plus magnesia ( $\text{CaO} + \text{MgO}$ ), and these four constituents will form about 96 per cent of the total slag. Typical analyses will show  $\text{SiO}_2$ , 25 to 50 per cent;  $\text{Al}_2\text{O}_3$ , 5 to 20 per cent;  $\text{CaO}$ , 25 to 50 per cent;  $\text{MgO}$ , 0 to 25 per cent.

**584. The Efficiency of the Blast Furnace.**—As a utilizer of heat the modern blast furnace is one of our most efficient types of furnace. The heat balance of a furnace of 550 tons daily capacity is tabulated on p. 541.

It will be noted from the heat balance sheet that account is made of the heat energy of the dry top gases due to their temperature only. If account is made of the combustion energy in the carbon monoxide in the top gas—9,000,000 B.t.u.—it appears that 33 per cent of the total available heat energy in the furnace is utilized in reducing impurities in the ore, that 19 per cent is lost through operations of a secondary character and that approximately 48 per cent passes out with the top gases. Of the latter amount about 41 per cent is usable. About 20 per cent of the total heat energy is needed to run the stoves, which having an efficiency of about 50 per cent return 10 per cent to the furnace. Consequently 31 per cent is available for power. With steam boilers and engines,

of this surplus is needed for the hoisting, pumping, and blowing requirements of the furnace. But with large plants and efficient gas engine installation, about 25 per cent of the total energy of the fuel is available as surplus power after plant requirements have been satisfied. This approximates 48,000 theoretical horse-power of surplus power for a furnace of the capacity considered. If the gas were used to drive gas engines about 15,000 horse-power could be developed.

## HEAT BALANCE SHEET FOR ONE TON OF PIG IRON

showing an efficient performance of a 550-ton blast furnace operating under a high temperature (673° C. or 1243° F.) blast

(W. Mathesius, *Bull. Am. Institute Min. Engr.*, Mar., 1915)

HEAT GENERATED.			HEAT CONSUMED.		
By.	B.t.u.	%	By.	B.t.u.	%
Combustion C to CO . . . .	3,995,000	31.1	Reduction of $\text{Fe}_2\text{O}_3$ . . . .	6,133,300	
Combustion C to $\text{CO}_2$ . . . .	6,754,000	52.6	Reduction of $\text{Fe}_2\text{O}_4$ . . . .	490,000	
Hot blast (heat content) . . . .	2,057,000	16.0	Reduction of $\text{MnO}$ . . . .	48,400	
Moisture of air in hot blast (heat content) . . . . .	42,000	0.3	Reduction of $\text{P}_2\text{O}_5$ . . . .	17,000	
			Reduction of $\text{SiO}_2$ . . . .	418,500	
				7,107,200	55.3
			Calcination of carbonates . . . . .	702,000	5.5
			Dissociation of moisture in blast . . . .	397,000	3.1
			Carried off with iron . . . . .	1,142,500	8.9
			Carried off with slag . . . . .	909,000	7.1
			Carried off with dry top gases . . . .	752,400	5.9
			Carried off with moisture in top gas . . . .	700,100	5.4
			Radiation, cooling water, and unaccounted for . . . . .	1,137,800	8.8
Total . . . . .	12,848,000	100.00	Total . . . . .	12,848,000	100.00

## CHAPTER XVIII

### MANUFACTURE OF WROUGHT IRON AND STEEL

**585. Introduction.**—Of the total annual production of iron and steel in the United States, about 80 per cent is steel and the remainder cast iron. Although direct production of the steel from the ore is feasible, and seemingly would be advantageous, in view of the proportionate magnitude of its consumption, economic considerations have determined that present practice should reduce practically all of the ore to the form of crude pig or cast iron, as one step in the manufacturing process; then convert this product into steel by refining or purification as the second step. Typical analyses of crude and finished product are as follows:

	C	Si	S	P	Mn
Pig iron .....	3.5-4	1-3	0.03-0.12	0.05-1.0	0.50-1.0
Steel, .....	0.10-0.75	0.05-0.20	0.03-0.07	0.03-0.10	0.30-0.75

Service requirements of strength and ductility necessitate the reductions of the amounts of the elements in the pig iron, in particular the carbon, silicon, and at times the phosphorus, to the lower limits prescribed by experience.

**586. The Purification of Pig Iron.**—Oxidation is the essential chemical principle upon which processes for the conversion of pig iron into steel are based. The accompanying tabulation shows the principal reactions and the quantities of heat thus generated.

Reactions.	CALORIES OF HEAT GENERATED.	
	Per Kg. of Metal.	Per Kg. of Oxygen.
$\text{Si} + \text{O}_2 = \text{SiO}_2$ .....	6428	5625
$\text{Mn} + \text{O} = \text{MnO}$ .....	1653	5681
$\text{C} + \text{O} = \text{CO}$ .....	2430	1823
$\text{Fe} + \text{O} = \text{FeO}$ .....	1173	4106
$\text{P}_2 + \text{O}_5 = \text{P}_2\text{O}_5$ .....	5703	4562

Silicon will unite with oxygen to form  $\text{SiO}_2$ , and manganese to form  $\text{MnO}$ , both of which products are insoluble in the molten iron, and because of lower specific gravity, will rise to the top as slag. The silica ( $\text{SiO}_2$ ) will in turn unite with the manganese oxide ( $\text{MnO}$ ), or any other basic constituent, such as iron oxide ( $\text{FeO}$ ) or lime ( $\text{CaO}$ ), to form stable silicates of a more fusible character than the constituent radicles. Carbon will oxidize to carbon monoxide ( $\text{CO}$ ) (or to  $\text{CO}_2$  at times) which will bubble through the bath and escape as a gas. The relative order of affinity for oxygen is about as given in the above list of reactions. After the silicon, manganese and carbon have been eliminated, iron will oxidize to  $\text{FeO}$ , a very small proportion of which is soluble in the molten iron; the greater part, however, rises to the top of the bath and becomes a basic constituent of the slag. Phosphorus will oxidize freely to the gas  $\text{P}_2\text{O}_5$ ; but under the conditions prevailing in the steel-making operation, this element will not be eliminated unless it is locked up in the stable form of calcium phosphate  $(\text{CaO})_4\text{P}_2\text{O}_5$ . To accomplish this end, lime additions to the bath are necessary, and form the essential feature of the basic process, which will be discussed more fully later.

The oxygen needed in the reactions may be obtained from the air; or it may be gotten by adding  $\text{Fe}_2\text{O}_3$  to the bath, in which case the oxygen is seized by the silicon, manganese, or carbon, and the liberated iron becomes part of the resultant steel. A significant feature in the tabulation of reactions given above, is the heat liberation accompanying each. All of the elements which it is desired to eliminate from the iron have a fuel value, in most instances greater than that of carbon itself. With proper details of combustion these fuel values may be utilized not only to supply the heat dissipated during the interval of refining a molten bath of cast iron, but in addition to furnish the increment of heat needed to raise the temperature the several hundred degrees necessary to ensure fluidity in the final steel. To illustrate: A ton of coal may be burned so slowly and inefficiently that there will be barely appreciable liberation of heat and a maintenance of temperature hardly sufficient to ensure continuous ignition and combustion. On the other hand, by rapid combustion with forced draft in an efficient type of furnace, heat liberation and temperature is such that iron or other refractory substances may be melted. The total quantity of heat is in both instances the same; temperature rise is the result of rapidity of generation and effectiveness of utilization. If oxidation of the elements in a bath of pig iron is sufficiently rapid, temperature rise may result; if slow, rate of heat generation may not equal that of its dissipation, and fuel from other sources may be required to keep the bath molten.

## THE PUDDLING PROCESS FOR MAKING WROUGHT IRON

**587. History.**—Modern steel-making methods were preceded by the puddling process, invented by Cort in 1784, improved by Hall in 1830, and persisting to the present day, although it went into the background with the inception of the Bessemer process and the "Age of Steel," in 1856. Puddled iron is to-day about 3 per cent of the annual output of iron and steel.

**588. Operation of Process.**—The puddling operation is carried out in a reverberatory furnace with rather small hearth and large grate area to ensure the working temperature desired. A longitudinal section of such a furnace is given in Fig. 1.

The furnace has a dish-shaped hearth consisting of a cast-iron bottom plate on which is fused about two to three inches of protecting cinder, essentially oxide of iron. The sides and ends are lined

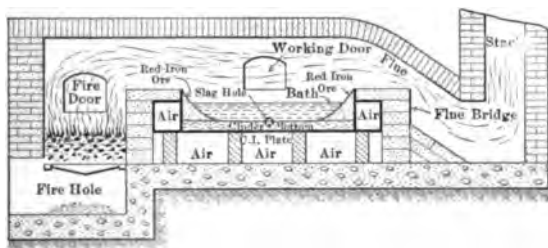


FIG. 1.—Showing Principle of Puddling Furnace.

with iron ore. Side doors are provided for handling the charge and product, and for rabbling the bath during the refining operation. The furnace has a low-hanging arched roof of refractory brick. The fuel is usually bituminous coal, preferably of low sulphur percentage and of high volatile content to ensure a long flame. Heating of the bath is accomplished entirely by the temperature of the gases and by their combustion over its surface. The products of combustion escape to a stack, necessarily at high temperature. To save some of the heat, waste heat boilers are often installed above the furnace, and the furnace gases pass through the boiler flues before they enter the stack. Natural draft is used, and there is no preheating of the air used for combustion.

The capacity of puddling furnaces ranges from 300 to 1500 lb. per heat; in general about 600 lb. Pig iron of white or mottled grade, and known as forge or mill iron, is charged on the banks of the hearth, together with some old slag or iron oxide to furnish a slag blanket for the bath, and to serve as the basic constituent of the new slag and as oxidizing agent in the refining.

A single heat takes about one and a half hours. After melting of the charge, iron oxide additions are rabbled into the bath and serve to remove practically all of the silicon and manganese and some of the phosphorus into the highly basic iron silicate slag, thus formed. Carbon





The puddling process has remained in the background in producing importance, and has undergone but little development, largely because of the high labor cost, small output, and heavy fuel cost. Some mechanical puddling furnaces were brought out, and perhaps might have been developed to a fairly high degree of efficiency, were it not for the ascendancy of the present-day steel-making processes.

**589. Kinds of Wrought Iron and Their Uses.**—The pure muck-bar or puddled wrought iron above described is in a considerable demand for wrought iron for blacksmith stock, because of its good welding properties, and for welded pipe and sheet metal, where resistance to corrosion is a factor. It is also used as a base for high-grade tool steels.

Besides pure muck-bar iron the following three varieties of wrought iron are commonly marketed: *fagoted iron*, *busheled iron*, and *charcoal iron*.

*Fagoted iron* is hot rolled from box piles the outsides of which are formed from muck bars or soft steel plates and the interior of scrap wrought iron or soft steel. If the stock is all wrought iron a homogeneous product of good quality may result, but if there is much scrap steel in the box piles the iron is likely to be of poor quality. This product is often made into merchant bars and plates.

*Busheled iron* is made in much the same way as muck-bar iron excepting that the metal charged is wrought-iron scrap, steel scrap or a mixture of both. On account of the variability in the composition of the scrap it is not as homogeneous as puddled iron. Busheled iron is considerably used in making plates and sheets where forging and welding properties are of minor importance.

*Charcoal iron* is made either from charcoal pig iron or from wrought-iron and steel scrap. Charcoal pig iron is made by remelting ordinary pig iron in a hearth heated by charcoal. It is much used in Sweden for the production of the famous high-grade wrought iron of that country. In America most of the charcoal iron is made by remelting wrought-iron and steel scrap in contact with a charcoal fire. This is generally done in a knobbling or sinking fire where the metal is charged on the bottom of the hearth and the charcoal fuel placed above it. During smelting the furnace-man continually lifts the metal to permit oxidation of impurities by a strong air blast. The latter is admitted through a tuyere placed on the side of the hearth. Knobbed charcoal iron is freer from slag and more ductile than ordinary puddled iron. It is surpassed, however, in both respects by Swedish charcoal iron. Knobbed charcoal iron is used in making crucible steel and is a good material for stay-bolts, engine-bolts and other forms requiring a tough, ductile metal with high resistance to corrosion. It also possesses excellent forging and welding properties.

## STEEL MAKING

**590. Classes of Processes.**—The prominent present-day steel-making processes are as follows:

Bessemer or Pneumatic	{ acid basic or Thomas-Gilchrist.
Open-hearth or Siemens-	{ acid Martin
	{ basic.

The distinctive difference between Bessemer and open-hearth processes is essentially one of type of apparatus to effect the conversion; while the acid and basic methods, adaptable to either type of equipment, differ in the chemical reactions involved, and in the resultant character of refining an elimination which may be effected.

The fundamental difference of acid and basic processes is the type of slag; in the former, slags of acid character, that is, high in  $\text{SiO}_2$ , or with the silica unsatisfied, are formed. And since any unsatisfied silica in the slag would tend to reach saturation by attacking any bases with which it might come into contact at high temperature, the refractory lining of the furnace is made of acid material (silica, gannister, mica schist, etc.) to prevent scouring. With acid slags there is no absorption and holding of phosphorus which might tend to be oxidized during refining. The acid process will not eliminate phosphorus.

The basic process provides basic slags, usually by lime additions in sufficient quantity to more than satisfy the silica and to have the basic constituents predominate. This can only be ensured by lining the furnace with basic refractories (magnesite, dolomite, etc.) since any excess base would tend to neutralize itself by scouring an acid lining with which it might come in contact at high temperature. With excess of lime in the slag, phosphorus after oxidation to  $\text{P}_2\text{O}_5$ , unites with such lime to form the stable phosphate  $(\text{CaO})_4\text{P}_2\text{O}_5$ , which in turn is absorbed by the slag. The basic process will eliminate phosphorus; likewise by proper conditions of working, there is a slight oxidation of sulphur, and some further elimination as calcium sulphide.

#### THE BESSEMER PROCESS

**591. Principle of the Process.**—The Bessemer process was introduced in 1856. It depends upon the essential principle of sufficiently rapid oxidation of the elements—Si, Mn, and C—which it is desired to remove from the iron, to keep an initially molten bath fluid throughout the whole of the refining operation, and deliver a molten product. In such case, because of lack of mutual solubility and differences of specific gravity, the slag separates from the steel by flotation, and the latter (unlike wrought iron) is free from this constituent.

**592. The Converter.**—A cross-section of the Bessemer vessel is shown in Fig. 2. It consists of a steel shell in three detachable sections—bottom, body, and nose. The body section is cylindrical, while the nose tapers to a relatively small opening, in order to conserve heat, prevent excessive ejection of metal and slag, and to provide for proper handling of metal in charging and pouring. The nose section may be concentric with the body axis, or set eccentrically. The several sections are held together by means of stirrups and wedges, in order that they may be readily separated

in spite of heat and slag accumulations. The shell is supported on trunnions, one of which is hollow to allow of the blast being kept on regardless of the position of the converter. A pipe leads from the hollow trunnion to the wind box at the bottom of the vessel. The converter is tilted by means of a pinion mounted on the trunnion, and rotated by a hydraulically operated rack.

Bessemer vessels are built in sizes from  $\frac{1}{2}$  ton to 20 tons capacity per heat; in general 10 to 15 tons represents the customary steel works size. In acid working, the body and nose are lined with a 12-in. thickness of rammed ganister, or blocks of mica schist or other silicious material. The bottom section is a pan, with rammed silica lining about 2 ft. thick. In

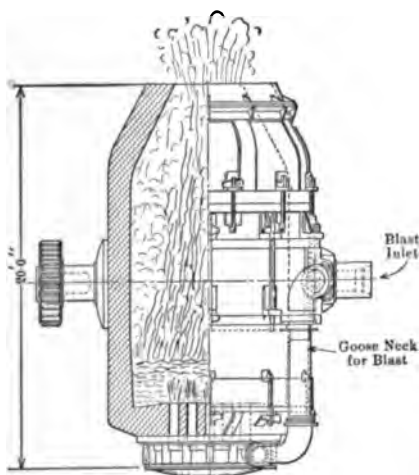


FIG. 2.—A 15-ton Converter.

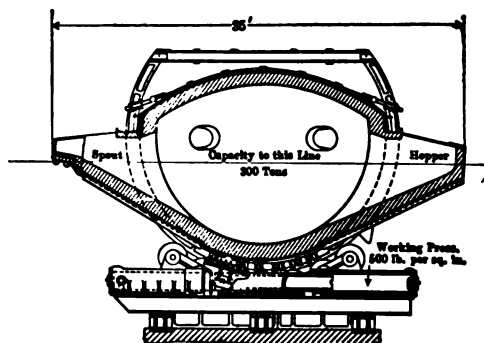


FIG. 3.—A 300-ton Mixer.

this bottom are set 15 or 20 silica refractory tuyeres, each provided with a dozen or more holes from  $\frac{1}{4}$  to  $\frac{3}{8}$  in. in diameter. A clay joint is made in keying the bottom to the body, to prevent leakage of metal. The body lining lasts for 10,000 to 20,000 heats, or a period of several months. The bottom is worn away in 20 to 25 blows, or a period of five or six hours. This is due to the fluxing action of the iron oxide formed at the bottom immediately upon entrance of air through the tuyeres. Facilities are provided for rapid change of bottoms.

**593. The Acid Bessemer Process.**—The charge for the converter must be molten. The pig iron may be melted in cupolas, but is now usually brought in ladle cars from large mixers or reservoirs of 200 to 600 tons capacity (Fig. 3), the mixer in turn taking its hot metal from the blast furnace. In this way there is no confusion or delay in working because of the large casts of the blast furnace at six-hour intervals, and the rela-

tively small charge requirements of the converter at fifteen-minute intervals. Also, irregularities of composition of the different blast-furnace casts are equalized for the Bessemer heats. In charging, the converter is tilted to a horizontal position, and the hot metal charge poured in; the blast is then put on, the vessel turned upright, and the heat blown. The blast pressure is about 20 to 25 lb. per square inch in bottom blowing and the metal is kept out of the small tuyere openings by this pressure.

The time of blowing is about twelve minutes; for the first few minutes silicon and manganese are chiefly oxidized, and form slag. During this interval there are iron sparks and slag ejections, but no real flame. Then carbon begins to oxidize in the converter to carbon monoxide, and burns at the nose of the vessel to carbon dioxide with a flame which increases in intensity, and is accompanied by the roaring boil of the bath as the gases go through it and agitate it. After several minutes, the flame flickers and drops, a sign to the blower that the carbon is practically burnt out of the bath. With the drop of the flame, the converter is turned down, the blast is cut off, and the metal is cast after the proper additions have been made.

In American practice, the blow is continued until practically all of the silicon, manganese and carbon of the bath have been eliminated. During the interval, iron oxide is formed, and in the absence of the above elements, there is no chance for its reduction to metallic iron at the end of the blow, and a small proportion remains dissolved in the iron of the bath. Also there is saturation of the bath in dissolved gases. A heat poured in this condition would result in rotten steel unsuitable for rolling or forging. The remedy is deoxidation with ferrosilicon and manganese alloy (ferro-manganese or spiegeleisen) added while teeming the steel into the pouring ladle. The silicon and manganese rob the iron of its oxygen, and separate as oxides (insoluble in the bath of steel) into the slag. The manganese, also, if in sufficient quantity, unites with sulphur to form manganese sulphide ( $\text{MnS}$ ), which remains in the ingot but in a form less conducive to red shortness than that resulting from iron sulphide. Final "killing" or quieting of the heat is effected by treatment with small amounts of aluminum thrown into the ingot mold during casting. The manganese alloys, which carry several per cent of carbon, serve also as recarburizers to bring the carbon content of the steel to the amount required by the specifications for varying character of service. The extent of conversion is about as follows:

	C	Si	S	P	Mn
Pig iron.....	3.5-4	1-1.25	0.05	0.09	0.50-1.75
Steel.....	0.10-0.60	0.05-0.20	0.05	0.10	0.30-0.50

The silicon and manganese values in the steel are the amounts of these elements remaining alloyed with the iron, in excess of the requirements for deoxidation. Carbon is added to the limits of specifications; sulphur and phosphorus are not removed during conversion, and are in the steel in greater amount than in the pig-iron charge, because of concentration due to the conversion loss during blowing.

The carbon in the pig iron is in fairly constant amount as it comes from the blast furnace; while the manganese content is largely a characteristic of the iron ore used. Silicon, being the essential fuel in converting, is held to close limits; below 1 per cent there is liability of cold heats, while with rapid and efficient blowing a silicon content much in excess of this amount only tends to prolong the heat and introduce difficulties through too high temperature during the blow.

During the heat there is a conversion loss of 8 to 10 per cent; 5 to 6 being accounted for by the silicon, manganese, and carbon removed; the remainder consists of a loss of iron, some because of sparks and splashing, but principally as iron oxide taken up by the slag to supply its deficiency in basic constituents. The amount thus required varies with the quantity of metalloids removed from the bath, with the composition of the slag, and with the amount of scouring of the bottom. No slag-forming constituents are added during the heat. The quantity of slag formed is about 150 lb. per ton of steel made, and has a composition approximately  $\text{SiO}_2$ , 50–70 per cent,  $\text{FeO}$ , 10–25 per cent,  $\text{MnO}$ , 10–30 per cent.

**594. The Basic Bessemer Process.**—The basic process was adapted to Bessemer working in 1876 by Sydney Thomas and Thomas Gilchrist. In basic Bessemer practice the vessel is the same as that used for acid working except for the lining of dolomite or magnesite to resist scouring by the basic slag. This lining is rammed into place with a tar binder. Phosphorus removal is effected during the blow by oxidation to the pentoxide ( $\text{P}_2\text{O}_5$ ) and subsequent formation of calcium phosphate  $(\text{CaO})_4\text{P}_2\text{O}_5$ , which becomes a constituent of the slag. The necessary lime is added to the converter before and during the heat. Silicon, manganese, carbon and phosphorus are eliminated in about the order named and the period of blow is from fifteen to twenty minutes.

Typical conversion is as follows:

	C	Si	S	P	Mn
Pig iron.....	3.5–4	0.5–1.0	0.05	2–3	1–1.5
Steel.....	0.10–0.60	0.20	0.04	0.05–0.10	0.30–0.50

The necessary heat is obtained during the blow from the oxidation of the silicon, manganese, carbon, and phosphorus. The silicon, how-

ever, must be kept low, because of its acidity and the increased lime needed to flux it. The deficiency in heat value thus resulting, together with the extra amount required to bring the high lime additions to slag fusion temperatures and to supply the increased radiation losses during a longer period of blow, are supplied by the phosphorus. The suitable pig iron is therefore one having a phosphorus content of 2 to 3 per cent, and is obtained from the ores of special districts, notably in Germany. Basic Bessemer working is not practical in the United States, although many of our Southern ores are suitable. The conversion losses are from 12 to 17 per cent, about 9 or 10 per cent of which is accounted for by the metal-loids eliminated. The slag weight will be 400 to 500 pounds per ton of steel, with a composition of  $\text{SiO}_2$ , 5-15;  $\text{P}_2\text{O}_5$ , 15-20;  $\text{CaO}$ , 40-45;  $\text{MgO}$ , 0-10;  $\text{FeO}$ , 5-20;  $\text{MnO}$ , 5 per cent. Because of its high content in phosphoric acid, basic Bessemer slags have value as a fertilizer after pulverizing.

**595. The Tropenas Converter.**—A modification of the standard bottom-blown converter used in steel works is the side-blown vessel of which the Tropenas converter is a type. The air is introduced through the side of the vessel at the surface of the bath under a pressure of 3 to 5 lb. These converters have a capacity of  $\frac{1}{2}$  to 2 tons per heat, and are used chiefly for the manufacture of steel castings, where hot steel free from occluded gases is particularly desirable. The lessened agitation of the bath in side blowing favors the latter, although accompanied by increased conversion loss and time of blowing. By allowing part of the tuyeres to deliver free air above the surface of the bath, the initial carbon monoxide is burned to carbon dioxide, and the full calorific value of each unit of carbon ( $\text{C}$  to  $\text{CO}_2$  gives 97,200 calories against 29,160 produced in burning  $\text{C}$  to  $\text{CO}$ ) is utilized within the body of the converter and the temperature of the bath raised thereby. The molten metal is obtained from cupolas, and must have a silicon content of 2 per cent or upwards in order that sufficient heat may be generated in the small vessel to maintain fluidity.

#### THE OPEN-HEARTH PROCESSES

**596. Essential Features and the Development of the Processes.**—The fundamental distinction of the open-hearth process as compared with the Bessemer is the relatively slow rate of oxidation of the impurities; so slow that the heat of reaction is insufficient to keep the bath fluid during conversion, and additional heat from outside sources becomes necessary to insure completion of refining and final fluidity of the steel and slag. Of necessity any coal or coke used as fuel must be in a compartment of the furnace separate from that of the bath, to enable desired oxidation reactions to be carried out. Combustion of coal in a separate grate box will hardly give the high temperature required for fusion of the finished steel, as was noted in discussion of the puddling furnace. The

inception of the open-hearth process was marked by the invention of the gas producer by Sir William Siemens in 1862, the utilization of the gas to obtain high temperatures by preheating of the gas and air required for combustion by regeneration, and the development of these principles and their application to the furnace and methods for refining of the steel between the years 1862 and 1868. Siemens method was the use of all pig-iron charges, with iron ore as the agent to supply the necessary oxygen. About the same time the application by Pierre Martin of the Siemens furnace to the manufacture of steel by dilution of pig iron with scrap, and subsequent refining, caused his name to become identified with the development of the open-hearth process; it is still known in Europe as the Siemens-Martin process.

**597. The Open-hearth Furnace.**—The modern open-hearth furnace is built in capacities of 5 to 100 tons per charge and may be stationary or tilting, the former being the more usual.

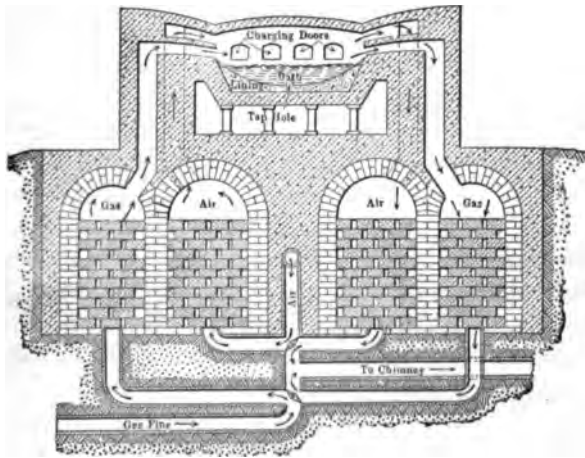


FIG. 4.—Vertical Section through a Stationary Open-hearth Furnace Showing Principle of Operation.

(The regenerators for gas and air are below and behind the hearth.)

The smaller sizes are used mainly for the manufacture of steel castings, with usual capacities of 15 to 25 tons. The furnace hearth consists of a shallow steel pan, supported by foundation arches, and lined to a thickness of about 2 ft., with brickwork over which are successive layers of refractory sintered into a continuous bottom.

For acid practice, this bottom is made of silica brick and sand; for basic working, of magnesite brick and crushed magnesite or dolomite with tar binder. Fig. 4 shows a longitudinal vertical section of a stationary open-hearth furnace. Fig. 5 shows a transverse section through an open-hearth furnace house.

A 60-ton furnace has hearth dimensions of about 40 ft. in length by 15 ft. wide, with a depth of metal of 18 to 24 in. Length is limited largely by necessity of high and uniform temperature throughout the bath; width by necessity of effective patching of bottom; while too great a depth means inefficient treating of a charge which must be heated entirely from the top surface.

The furnace is enclosed on sides and top by walls and a low hanging arched roof of silica brick, secured by buckstays and tie rods. Above the hearth there is no contact with slag; therefore there is no necessity for using the more expensive and mechanically weaker magnesite bricks, even in basic practice. Charging is done through doors at one side of the furnace, while in the middle of the opposite side the bottom slopes to a tapping hole. The level of bath and charging floor is usually at sufficient height above the pouring floor to enable ladles to be set under the tapping spout without necessity of deep ladle pits. This arrangement also enables flues, valves, and regenerators to be placed in accessible positions under the charging platform.

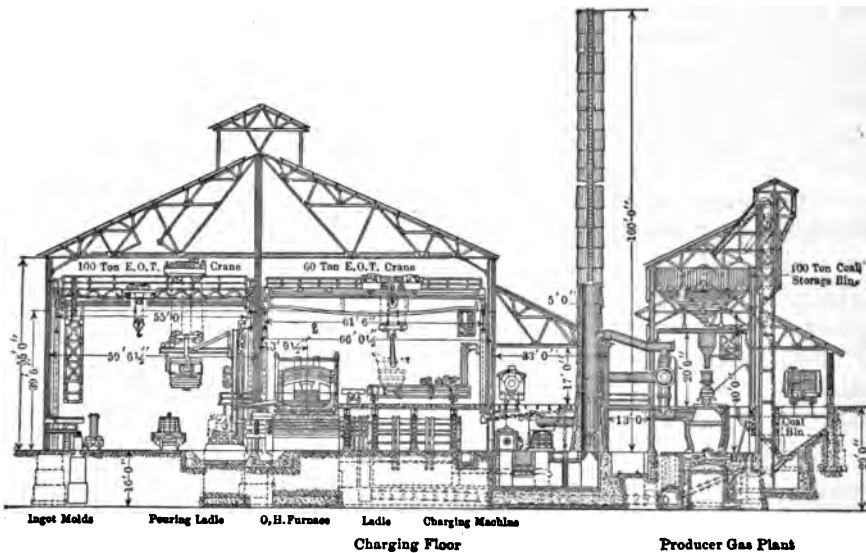


FIG. 5.—Transverse Section of an Open-hearth Plant at South Works of the Illinois Steel Co., Chicago.

At each end of the furnace are separate uptake flues for the gas and air used for combustion, and ports to direct them into the furnace for proper mixture and combustion. The uptake flues lead into heat storing or "regenerator" chambers one each for gas and air at each end of the furnace. These chambers are usually built under the charging floor, and are filled with built-up rows of a checker work of silica brick, so piled as to allow of fairly free passage of the gases, and yet present a large surface of contact for heat absorption and extraction. Between the regenerator chamber and the single stack for each furnace is a set of flues and hydraulically operated valves by which the combustion gases and waste gases can be led to and from either end of the furnace, and reversed periodically as desired. Natural draft is employed.



Producer gas is the customary fuel, although natural gas has played a very important part in American practice, and crude oil, pulverized coal and tar have been used. For other fuels than producer gas only the air required for combustion is preheated, and no regenerators are required for the fuel. In the use of producer gas, both gas and air are heated prior to combustion. The fuel and air ignite at the ports at one end of the furnace, and burn in their passage over the surface of the bath of metal, escaping at the other end at high temperature, approximately that of the furnace itself. Passing on their way to the stack through the regenerator chambers at that end of the furnace, these hot gases give up a large part of their heat to the brick checkerwork. This brickwork stores up a large quantity of heat and in an interval of about twenty minutes has attained a high temperature. The direction of flow of gases is now reversed, and combustion is at the opposite end of the furnace hearth. The entering gas and air, passing through the hot regenerator chambers, absorb their stored up heat, and reach the ports at a temperature of  $1000^{\circ}\text{C}$ . The temperature attained in the furnace is now greater than that due to direct combustion of the fuel and air, by an increment dependent upon the heat stored up in these gases. The waste gases in turn give up their heat to the cooler regenerators at the opposite side of the furnace. The periodic reversals at intervals of fifteen to twenty minutes enable the necessary temperature to be maintained.

**598. Smelting by the Open-hearth Furnace.**—Usual open-hearth practice is with basic bottoms; in the United States the relatively small tonnage of acid steel is used largely in castings.

The charge may be cold or molten; in the latter case there is some saving of fuel and time. Also since the success of the process does not depend upon the fuel value of the elements in the burden, the latter may vary widely in composition. Usually the charge is a mixture of pig iron and steel scrap; increase of the last named effects partial refining of the bath by dilution.

The reactions are virtually the same as in Bessemer working; an oxidation of the silicon, manganese, and carbon; and in basic practice, an elimination of phosphorus by oxidation and union with lime. The rate of reaction is, however, very slow, and heats require from six to twelve hours, usually about ten hours for basic heats of 50 to 75 tons each. The oxidizing agent is iron ore; by interaction the oxygen of the ferric oxide ( $\text{Fe}_2\text{O}_3$ ) unites with the metalloids and is eliminated as gas or slag; the liberated iron in turn becomes a part of the bath. With ore additions it is possible, therefore, to practically offset the conversion loss due to elimination of metalloids, and to keep it to 5 per cent or less. Basic open-hearth slags in this country rarely carry sufficient phosphoric acid to warrant their use as fertilizer; this is due to the moderate phosphorus

content (0.20 to 1 per cent) in the charge. Control of the heat is by fracture tests of samples and by chemical analyses of same; the heat is tapped under controllable conditions. Ferromanganese and ferro-silicon additions are made to the ladle on tapping, and aluminum is thrown into the ingot mold as in Bessemer practice.

After each heat the furnace bottom is patched by shoveling in crushed and calcined dolomite, which frits into place at the working temperature. The lining and bottom are good for three to six months of service, when shut down for repairs and rebuilding is necessary.

**599. Comparisons of Bessemer and Open-hearth Processes.**—Factors of advantage and disadvantage in the Bessemer and open-hearth processes are as follows:

**BESSEMER**

Rapid conversion—10 to 20 minutes.  
 Relatively small heats—1 to 20 tons.  
 Small plant cost per unit of output.  
 All fuel is in charge.  
  
 Charge must be molten.  
 Little liberty in composition of charge.  
 Control entirely in skill of operator.  
 Heavy conversion loss—10 to 15 per cent.  
 Simplicity of operation.

**OPEN-HEARTH**

Relatively slow conversion—6 to 12 hr.  
 Relatively large heats—25 to 100 tons.  
 Heavy plant cost per unit of output.  
 Extra fuel required—50 lb. of coal per ton hour.  
 Charge may or may not be molten.  
 Much liberty in composition of charge.  
 Accurate control of product.  
 Small conversion loss—0 to 5 per cent.  
 Operation more complex than Bessemer.

In the United States, steel-making practice is divided almost entirely between the acid Bessemer and basic open-hearth methods. Comparative outputs in the United States, Germany and Great Britain in recent years are given in Table 1:

**TABLE 1.—TONNAGE OF STEEL INGOTS AND CASTINGS PRODUCED FROM VARIOUS PROCESSES IN LEADING COUNTRIES**

(In Long Tons)

Process.	1913.			1915.		
	United States.	Germany (Metric Tons).	Great Britain.	United States.	Germany (Metric Tons).	Great Britain.
Acid Bessemer. . .	9,545,706	155,138	1,048,772	8,287,213	165,290	821,408
Basic Bessemer. . .		10,629,697	551,929		6,528,146	479,816
Acid open-hearth. .	1,255,305	380,155	3,811,382	1,370,377	243,111	4,090,752
Basic open-hearth. .	20,344,626	7,592,901	2,251,793	22,308,725	5,376,931	2,958,968
Crucible. . . . .	121,226			113,782		
Other. . . . .	34,011*			70,939*		
<b>Total. . . . .</b>	<b>31,300,874</b>			<b>32,151,036</b>		

\* Principally from electric furnaces.

**600. The Decline of the Bessemer Process.**—In spite of the simplicity of the Bessemer process and its many other features of seeming advantage over the open-hearth method, the former has been steadily forced into the secondary position in this country. This is largely due to the gradual increase in the phosphorus content of available ores; the difficulty of securing ores of Bessemer grade has caused a spread of price between this and grades suited for basic working sufficient to make the open-hearth method economically attractive. In addition there is the advantage of greater yield in conversion, and the better reputation for quality which open-hearth steel holds in the opinion of users. Bessemer steel finds its principal outlet in steel rails, pipes and tubes, wire and wire rods, plates and sheets, and merchant and agricultural machinery shapes. Practically all structural steel has for years been of open-hearth grade, and in recent years the latter has been rapidly superseding the Bessemer process in the manufacture of steel for rails.

**601. The Duplex Process.**—In order to conserve the advantages of the Bessemer process and existing plants for its manufacture, and yet to effect an elimination of phosphorus, the duplex method has been adopted with considerable success. A heat is blown to virtual elimination of silicon and manganese, and partial removal of carbon. It is then transferred to a basic open-hearth furnace, where the phosphorus is eliminated and the heat slowly finished to the desired quality. In certain plants electric furnaces are being used for the finishing operations in the duplex combination with the Bessemer converter. The duplex method effects a marked saving in the time necessary for treatment in the open-hearth furnace.

#### MINOR PROCESSES USED IN MAKING STEEL

**602. The Cementation Process** was the only known method of making steel until A.D. 1500. The process is still used in England for making cutlery and tools but has never found favor in America. The well-established fact that wrought iron packed in charcoal and heated to a bright red will absorb carbon and form a solid solution with it is the basic principle of the process.

For this process the cementing furnace is provided with a pair of long rectangular converting pots made of fire-brick and externally heated by a central fire-box. Long thin bars of the very best quality of Swedish charcoal wrought iron are packed in fine charcoal within the converting pots. The top of the pot is covered with a mixture of grindstone dust, the fire is started, and the temperature gradually raised until it reaches approximately 1000° C. A uniform heat is then maintained for a week or ten days, depending upon the amount of carbon desired in the steel. The carbon content is determined from time to time by withdrawing sample bars and testing them. After the proper carbon content has been secured the furnace is gradually cooled down in another week and the metal is withdrawn.

Since carbon monoxide is evolved during the process the surfaces of the bars are covered with blisters due to the combination of the carbon with the iron oxide in the

slag of the wrought iron. Furthermore the impregnation of carbon varies with the distance from the surface of the steel. In order to render the blister steel more homogeneous the bars are sheared into short lengths, piled, heated to a welding heat and rolled down into small bars called single-shear steel. Sometimes the single-shear bars are treated like the blister-steel bars and thus made into double-shear steel.

**603. The Crucible Process** was invented in A.D. 1740 by Daniel Huntsman of Sheffield, England. The process consists in melting wrought iron together with charcoal and a little ferromanganese in a small barrel-shaped vessel called a crucible. In America, steel scrap occasionally forms a portion of the charge and ferrosilicon is added just before drawing the crucibles. For very superior grades of steel Sheffield makers charge cementation steel instead of wrought iron. Various alloy steels are also made by this process. Thus ferrochromium and ferrotungsten are added to the charge in making chrome and tungsten steels, respectively.

Since neither sulphur nor phosphorus are removed in the process only very pure stock can be used. Crucibles are made of clay, or graphite and clay, and usually hold from 50 to 100 lb. Most of the melting furnaces are gas fired and are built upon the regenerative principle like the open-hearth furnace (Art. 597). Each furnace is provided with two to twenty separate melting holes. In many of the English furnaces coke fires are built in the melting holes in direct contact with the crucibles.

In American practice four to six crucibles are charged with stock fitted with tight covers, placed in a melting hole, and gradually heated to the melting point in two to three hours. Formerly crucibles were held in the furnace ("killed") for a half hour or more to allow the gases to boil out of the metal, present practice favors the introduction of ferrosilicon and ferromanganese at this juncture which in four or five minutes deoxidize and quiet the metal, thus avoiding a "killing" period. The crucibles are withdrawn as soon as the metal has become quiet (evolves no more gas), the slag is skimmed off and the metal poured into small cast-iron ingot molds or into special forms for steel castings. Ingots are rolled, forged or pressed, as in other steel-making processes, to densify the metal and improve its mechanical properties.

On account of the high cost of labor, fuel, and crucibles, in proportion to the quantity of steel produced, crucible steel is much more expensive than Bessemer or open-hearth steels. Consequently its use is limited to articles of small weight or to special parts where a high-grade product regardless of cost must be used. It is the principal source of steels used in making tools, cutlery and springs and also finds outlet for fine grades of cast steel. The annual output of crucible steel in the United States is from 100,000 to 150,000 tons.

**604. The Electrical Furnace in Steel Making.**—Within the last decade rapid strides have been made in the use of the electric furnace both in

super-refining steel and in purifying cast iron. In 1917 there were 136 electric furnaces in the United States and the production of electric steel for 1916 was in the vicinity of 170,000 tons. The electric furnace is somewhat similar to a small open-hearth furnace with electricity instead of gas as a source of heat. The principal advantages of the electric furnace process are: (1) the possibility of securing any desired temperature thus providing a means of removing impurities and occluded gases without perturbation of the bath; (2) the opportunity for changing slags thus rendering possible the removal of various impurities; and (3) high temperature with controllable atmosphere (oxidizing or reducing) as desired.

Because of the great expense in utilizing electricity as a source of heat energy, the electric furnace cannot be efficiently used to purify pig iron unless electrical energy can be purchased at very low rates. In steel making, the electric furnace is principally used to reduce the oxygen, phosphorus and sulphur from steel which has been partially purified by the acid Bessemer process. Superior grades of steel for rails, axles, castings and wire can be made in this way. It is also used to super-refine steel from the basic open-hearth furnace and thus make a high-grade steel which compares very favorably with the best crucible steel in quality, and costs less.

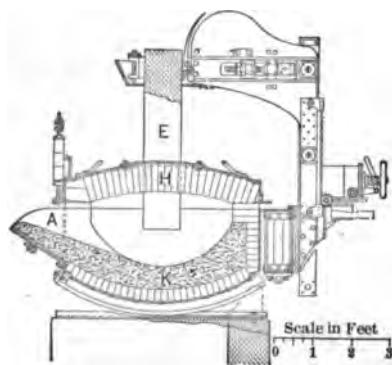


FIG. 6.—Transverse Section of Héroult Electric Furnace.

A, spout; E, electrode; H, roof; K, hearth lining.

The three types of electric furnaces used in refining steel are (1) the radiation arc type in which the current arcs directly from the positive to the negative electrode, both being suspended above the bath; (2) the arc resistance type in which the current arcs from positive electrode to bath and arcs or is conducted from the bath to negative electrodes; and (3) the induction type in which the bath of metal forms the closed secondary circuit in a step-down transformer. Three-phase alternating current is generally used in these furnaces. A Héroult furnace of Type No. 2 has been built with a capacity of 25 tons per charge. On account of its efficiency and

adaptability the Héroult furnace is used more than any other electric furnace for refining steel. Fig. 6 shows one of the small single-phase Héroult furnaces. In the larger furnaces three-phase current is used and the three electrodes are set over the bath on the apices of an equilateral triangle. In refining hot metal the larger Héroult furnaces consume from 150 to 200 kilowatt hours per ton of poured metal and require about 1 to 1½ hours for purification. With cold metal the time required is three to four times as long and energy consumption is from four to six times as great.

## CHAPTER XIX

### THE MANUFACTURE OF IRON AND STEEL SHAPES

**605. Essentials in the Production of Shapes.**—Iron and steel finds outlet in engineering construction in shapes produced either by casting or some form of mechanical working. In the former operation, the metal is melted and poured into molds made in sand or other suitable refractory substance; fluidity of metal is necessary, and it may be cast iron, steel, or malleable cast iron. Mechanical working of the metal, however, results in improved physical quality, and where this is essential, the required shape is produced by forging or rolling. For the latter operations malleability, at least while hot, is essential, and consequently cast iron is not suitable. Intricate or special shapes are produced by direct forging; or where numerous pieces of the same form are required, by drop forging with dies. By far the greatest tonnage of shapes required in engineering construction is of certain standard sections, uniform in cross-section throughout a length which is, in general, relatively much greater than the cross-section. Rolling mills can produce such shapes in quantity at a speed much greater and a cost much lower than any method of forging or casting.

**606. Ingots.**—Steel intended for rolling or forging is cast into ingots. These vary in shape and size, but in rolling mill practice are usually square or rectangular in cross-section, 12 by 12 in. to 20 by 20 in. or 18 by 24 in., and from 5 to 8 ft. long. The mold is of cast iron, 3 to 5 in. thick, with open top and bottom. The ingot mold is set upon end on an iron bottom plate, either stationary upon the casting floor, or on cars to enable prompt removal of the ingots after pouring. The mold has a gradual taper in cross-section from bottom to top, to facilitate freeing of the ingot. Steel is poured into the open top of the ingot mold from a large ladle, Fig. 1, with nozzle and stopper at the bottom, and handled by a crane. When the steel has solidified sufficiently, the mold is stripped from the ingot by lifting the former;

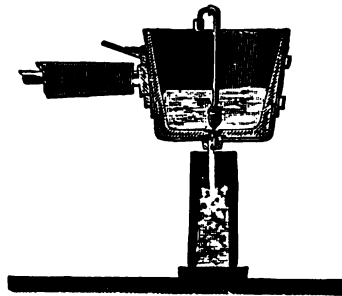


FIG. 1.—The Valvular Ladle.

or in case of sticking, by forcing downwards upon the ingot with a plunger at the same time that the ingot mold is lifted.

The following defects are present to some degree in all ingots: pipes, blow-holes, segregation, slag, and a very coarsely crystalline structure, termed ingotism. As the ingot cools the outside quickly forms a solid shell surrounding a molten interior. The shrinkage which accompanies the cooling of the interior produces a pipe. Blow-holes result from entrapped gases and from the solidification of certain impurities which shrink more than the iron in cooling. Segregation of the impurities is caused both from the rejection of impurities by the exterior as it solidifies and also by the differences in density, the impurities being lighter than iron. These defects are idealized in Fig. 2. Ingotism is produced through slow cooling from a very high temperature.

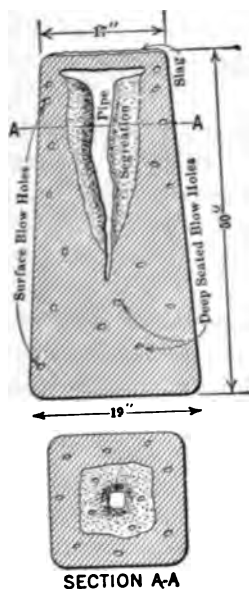


FIG. 2.—Defects in Ingots.

The head of the pipe, most of the slag and much of the badly segregated metal can be removed by cutting off (cropping) the upper third of the ingot. The evil effects of blow-holes and ingotism may be greatly reduced by proper mechanical treatment at a high temperature.

A long rectangular ingot cast on end makes a satisfactory form for rapid handling and for the initial roughing down in the rolling mill. Also, there is the tendency to concentrate the segregation of impurities and the pipe or cavity due to shrinkage in solidification to the last cooling portion at the center and top of the ingot. This defective portion can subsequently be cropped or sheared off during rolling to the proper amount to ensure sound steel.

**607. Heat Treatment of Ingots.**—Rolling or forging of the ingot immediately after stripping is not satisfactory. If the outside is at proper temperature, the inside is so hot that there is liability of squirting out of the fluid interior when pressure is applied. While if the interior is at rolling temperature, the exterior is too cold. The remedy is "soaking" of the ingot. Theoretically all that is required is a pit with efficient heat insulation so that interior and exterior temperatures of the stripped ingot can be equalized to the proper degree by diffusion. Actually, however, soaking pits are gas-fired furnaces equipped with regenerators. Thus the ingot can be brought to any desired temperature for rolling regardless of stripping conditions; also there will be a regular supply of ingots to satisfy the continuous requirements of the mill, despite the intermittent delivery

of the furnace; and no confusion, in case of accident or delay at either mill or furnace, will result.

**608. General Method of Rolling Shapes.**—When an ingot is inserted between two revolving rolls it is drawn through them in the direction of rotation, and at the same time there is downward and upward pressure upon the metal, and a pull in the direction of travel, which results in a reduction of thickness, a slight increase in width, and a material increase in length. A pair of simple rolls would in a single pass reduce an originally square bar to a rectangular one of even thickness equal to that of the distance between the rolls; the width, on the other hand, is slightly greater than the original, and of somewhat irregular and bulging outline, due to the downward pressure without side restraint. By a second pass through the same rolls, this time edge on, a square bar

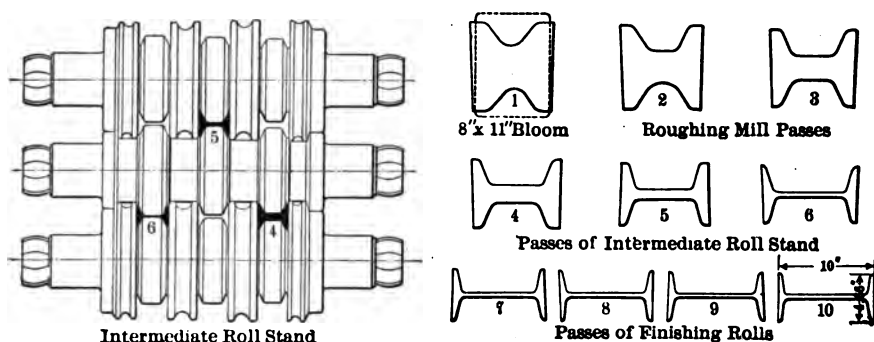


FIG. 3.—Typical Roll Passes in Forming a 10-inch I-beam. (Illinois Steel Co.)

is obtained of dimensions equal to the distance between rolls, and of increased length in proportion to the amount of reduction of original section. By successive passes through rolls of varying distances of separation, or by adjusting the center distance of a single pair of rolls, and by quarter turning, or edging the piece as required, a square or rectangular bar of uniform cross-section may be rapidly produced.

Or the same result may be obtained in rolls of fixed center distance, which have grooves turned in their surfaces of proper dimensions for the successive reductions. Again, by varying the character of the grooves, a wide variety of cross-sections can be produced. Fig. 3 illustrates types of rolls used in forming I-beams. It also shows the shapes assumed by a bloom as it is gradually reduced to an I-beam.

**609. Rolling Mills.**—Rolling mills have been developed from the original type with a single pair of rolls, or two-high stand. Such an arrangement has a limited capacity, since succeeding passes are made only by sending the piece back idle over the roll stand, with consequent loss of time and heat. The three-high mill has been adopted for the rolling



of most standard shapes. It consists of three rolls with their axes in the same plane, and so driven that by passing the bar through the lower pair in one direction, and through the upper pair on the return, reduction is obtained in both directions. For sections of great weight requiring very heavy mills with rolls of large diameter, the lift from lower to upper passes becomes impracticable; but rolling in each direction is accomplished by reversal of rotation of the two-high stand after each pass. In rolling rods or other shapes of very great lengths, there is much loss of time and cooling of the piece, even in three-high mills, if each pass is begun only after completion of the preceding one. By looping the rod back through the next pass as soon as the entering end comes through the rolls, several reductions may be effected simultaneously on the same rod, with great saving in time and heat. The capacity of the "looping mill" may be obtained by the "continuous mill." In this case the several stands of two high rolls are set in tandem, and properly geared to speed up successive passes to take care of the increased length of bar resulting from each reduction. The "universal mill" has a set of small rolls with their axes vertical; by varying the center distances of horizontal and vertical sets of rolls, a great variety of flat sections can be produced without multiplicity of rolls.

Ingot are rolled into blooms or billets, usually a rectangular or square form 10 by 10 to 2 by 2 in. in cross-section, on two-high reversing mills. The billets are sheared to the length required in the subsequent rolling operation, and are usually reheated.

Rolling mills have undergone much development, with resultant increase of output and elimination of hand labor. Heavy material is fed into the rolls by geared roller tables, and these tables in turn are made of the lifting, tilting, or transfer type to raise the bars from lower to upper passes or transfer them to others in different stands of the same mill train. The power required for driving varies from a few hundred to several thousand horse-power; heavy mills are usually driven by direct connection to steam engines, while electric motors have had much application for smaller mills, and especially for auxiliary equipment. The output of mills varies greatly. With light sections such as thin sheets and strips it may be as low as 10 tons per twenty-four hours; while certain mills will produce 4000 tons of billets or rails in the same period.

Most standard sections—rails, I-beams, channels, angles, round and square bars, etc.—are produced in three-high mills with rolls of fixed center distances and grooved to the special shapes.

The successive passes through the rolls from ingot or billet to finished section are given in Fig. 4 for typical shapes. In the design of the successive passes, the aim is to effect not only the reduction with maximum speed and efficiency, but at the same time to so shape the grooves or turn

the bar that work will be done as uniformly and, to as great an extent as practicable, upon all sides of the section, and thus obtain the beneficial effects of the mechanical treatment throughout the section.

Specialized lines of manufacture include plates, sheets, pipe and wire.

**610. Plates.**—Plates are of  $\frac{1}{8}$ -in. thickness and upwards. The ingot is rolled into a rectangular slab of cross-section sufficient to produce the length and thickness of plate desired. The slabs are cut to a length approximately equal to the width of the plate to be rolled. These slabs are reheated, and put through a two-high reversing or special three-high mill with flat rolls; the long side of the slab being parallel to the roll axes. The rolls are screwed down in successive passes until the plate has been reduced to the required gauge. After cooling it is straightened in a special set of rolls and sheared to size.

Rolled plates may be obtained up to  $2\frac{1}{2}$  in. in thickness, up to 132 in. in width, or up to 85 ft. long.

**611. Sheets.**—Sheets are thin plates from  $\frac{1}{16}$  to  $\frac{1}{8}$  in. in thickness. Billets are first rolled into sheet bars of rectangular section  $\frac{1}{2}$  by 8 to 1 by 10 in., on three-high mills. The sheet bars are cut to a length equal to the width of sheet to be rolled. After heating, a pair of bars are rolled at the same time, but separately, on a two-high mill with plain rolls, and with hand handling and screw downs to regulate thickness. After some reduction, the two pieces are rolled together, one on top of the other until too cold for further work.

From two to four sheets are then placed in a pack, doubled by bending transversely and stamping flat, and reheated. The pack of four to eight sheets is rolled to gage, after which it is sheared and opened. The sheets are annealed, and then given the various surface finishes or coatings required in service.

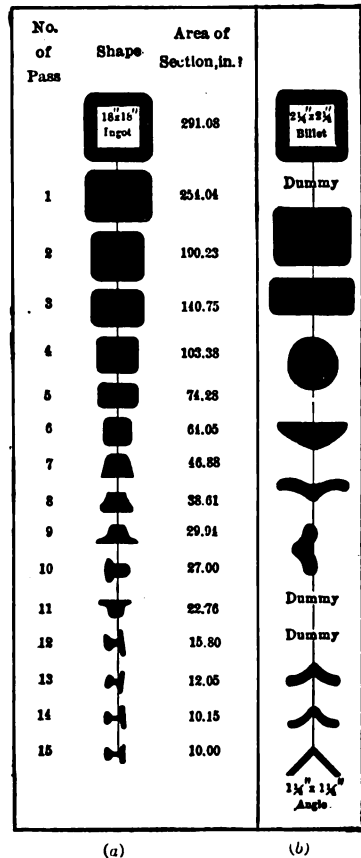


FIG. 4.—Changes of Shape in Rolling (a) an 100-lb. I-beam; (b) an  $1\frac{1}{2} \times 1\frac{1}{2}$  in. Angle. (*Iron Age*, Vol. 92, p. 968 and 1037.)

**612. Pipes.**—Pipes or tubes may be seamless, lap-welded, or butt-welded. Seamless tubes are made by piercing a round billet while it is being distorted by a set of special skew rolls; or by gradually pressing a plate into a closed end cup or tube. These hollow billets are drawn (hot or cold) to size over a mandrel and through dies. The principal outlet is for boiler tubes, bicycle tubing, etc., where maximum strength and lightness are desired.

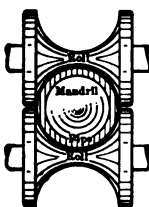
(1) SKELP WITH SCARFED EDGES



(2) SKELP ROUGHLY FORMED INTO PIPE



(3) WELDING BY ROLLING



(4) WELDING LAP-JOINT PIPE



(5) BELL THROUGH WHICH SKELP IS DRAWN TO FORM BUTT-JOINT PIPE

Fig. 5.

Butt-welded pipe is made from skelp which is welded along the butt joint without lap. The strips are heated to welding temperature; then drawn from the furnace through a bell mouthed-die, Fig. 5, which curls the strip to a circular form and forces the edges together with sufficient pressure to effect a continuous weld. The pipe is finished by passing through a pair of grooved sizing rolls. Butt-welded pipe is made in sizes from  $\frac{1}{4}$  to 3 in. in diameter; it is used for gas and water pipe and miscellaneous purposes where not subjected to great pressure.

**613. Wire.**—Rods are rolled in looping or continuous mills to about a  $\frac{1}{4}$  or  $\frac{3}{8}$  in. in diameter, coiled into bundles, and pickled in acid to remove scale. Further reduction is effected by cold drawing through dies of high-carbon steel. The drawbench consists of the drawplate and a power reel for pulling the wire through and coiling. Successive reductions may be

Lap-welded tubes are made from skelp, a flat strip rolled to proper thickness and of a width equal to the circumference of the pipe plus the lap, Fig. 5. The edges of the skelp are slightly beveled or scarfed to prevent too great an excess of metal at the lap. Up to 12 in. in diameter, bending to circular form is done by drawing the heated skelp through a bell-mouthed die; larger sizes are formed by putting the plate sideways through a set of three bending rolls. The formed pipe is then heated to welding temperature and passed through a pair of rolls with grooves, corresponding to the outside diameter of the pipe. The roll pressure is resisted by a mandrel or ball on the inside of the pipe which is mounted between the rolls and on the end of a long rod. The pipe is then sized in grooved rolls and finished and straightened with a pair of skew rolls. Lap-welded pipe is used for boiler tubes and for steam and hydraulic piping to withstand high pressure and is made in diameters  $1\frac{1}{4}$  to 30 in.

made on individual benches; or by multiple drawing in which case the coil is reeled only after reduction in several dies. A power reel is provided between each pair of plates around which the wire is given a couple of turns. After several reductions the wire becomes hard and brittle because of overstrain, and must be annealed before drawing can be continued. Speed of drawing is from 75 to 750 ft. per minute, depending on size and hardness of wire; reduction is from 20 to 25 per cent per hole.

**614. Forging and Pressing.**—Whereas it is only possible to roll members of uniform cross-section and for every change in cross-section it is necessary to provide a special set of rolls, forging provides a means of securing fine-grained, strong and tough parts of almost any required design. Forging requires little expense for special tool equipment but, on the other hand, on account of the length of time consumed does entail a much heavier fuel and labor charge than rolling. Since in forging the part is more completely under the control of the operator it is possible to finish it at just the right heat and to diminish the intensity of the blows in manner suited to securing a fine-grained product. The effect of the hammer blow, however, is not deep seated, consequently the process is best adapted to sections under 2 or 3 inches in thickness.

Small forgings may be made directly from ingots or from rolled bars. Ingots for small forgings are usually square or rectangular in form, those for large parts are cast in hexagonal sections with fluted surfaces. The latter type of ingot is less liable to crack in cooling than a square section. After cropping the remainder of the ingot is brought to a light red heat in a non-oxidizing flame and taken to the forge. Since the interior of a large ingot is often under high tensile stress due to cooling strains, reheating must be done at a very slow rate in order to avoid cracking. Often with ingots weighing many tons one or two days is required for reheating.

When the parts are of small size and the required number is small, reduction of the ingot or bar is accomplished by the hand hammer or sledge on the smith's anvil. If a large number of small parts are wanted *drop forgings* are made. They are formed between dies which are attached to the anvil and head of a steam hammer. In shaping intricate parts approach to final form is gradually made through the use of a number of sets of dies. Drop forgings are very commonly used in making various types of levers, wrenches, small connecting rods and crank shafts. They are superior to steel castings in quality on account of the mechanical work done upon them; but unless large quantities are wanted, they are more expensive due to high cost of dies.

Large forgings are reduced from the ingot under double-acting steam hammers which range in capacity up to 30 or 40 tons and are capable of striking 100 to 300 blows per minute. The ingot is clamped to a very heavy bar, called a porter bar, which is slung in an endless chain in such

manner that the forging may be pushed backward or forward under the hammer, or rotated as desired. In making heavy shafting it is good practice to bore out the central portion of the ingot before reheating thus rendering it easier to heat and reducing the probability of cracking. When the ingot is forged, a long bar fitted with a head like a torpedo, called a mandrel, is thrust into the hole and kept under the hammer. As the ingot is revolved and hammered on the outside the mandrel acts as an anvil on the interior. In this manner the metal of the hollow section is rendered much more compact and uniform in structure and properties than is possible with a solid section. Forging is usually stopped when the color becomes a dull red. If further forging is required the part is again reheated to a bright red.

*Pressing.* Armor plate, cannon tubes, heavy shafting and other thick heavy masses of steel requiring mechanical treatment are most effectively worked under heavy hydraulic presses. The action of the press is much deeper than that of the steam hammer or the rolling mill. Furthermore, the press effects a reduction in size of large parts more quickly than the steam hammer. Presses range in capacity from 500 to 14,000 tons and are operated under pressures of 800 to 8000 lb. per sq. in.

**615. Casting Steel.**—Steel castings are most effectively used for intricate parts which cannot be rolled or forged and which must possess high strength and toughness; also when the required number of parts is small and the cost of equipment prohibits rolling or forging. With proper care in manufacture, castings can be made having strength equal to the rolled or forged product but somewhat inferior in ductility and toughness.

Steel castings are made from metal smelted by the crucible, Bessemer, open-hearth or electric furnace processes. For small and medium size castings of the very best grade, the electric furnace, or the electric furnace in conjunction with an acid Bessemer converter or basic open-hearth furnace, is superior to all other processes. It is, however, less adaptable to a wide variety of work than the crucible process, which is commonly used for very small castings of all grades and qualities. For the production of very large castings and for obtaining a large tonnage of standardized castings of medium size with continuous operation, the acid open-hearth furnace is very efficient and turns out an excellent product. The basic open-hearth process shares the same field of production, but makes castings somewhat inferior in quality and less costly than those gotten from the acid open hearth. For intermittent operation and a variable tonnage demand, the acid Bessemer process is well adapted. Baby converters of the Tropenas type are often used where the output is small. The quality of acid Bessemer castings is in general inferior to that which can be gotten from the other processes.

Patterns for steel castings are commonly designed with an allowance  $\frac{1}{4}$  in. per foot for shrinkage. The castings are formed in green sand or

dry sand molds which are made in much the same way as those used for cast iron (Art. 747). With steel castings, however, a good deal of care must be exercised to properly vent both molds and cores, to provide adequate gates for rapid filling of the mold, and to install a sufficiency of properly placed risers in order that the interior of heavy sections may be kept full of hot metal until the whole is frozen. Chills are sometimes used to hasten solidification during pouring and reduce blow-holes.

Owing to the great shrinkage of steel in cooling, intricate castings, or those which vary considerably in cross-section, must be separated from the mold as soon as the metal has solidified in order to prevent cracking. After the castings have been cooled slowly under sand or in a heated furnace the sprues are broken or cut off and the surfaces are freed from sand.

Steel castings which are to be subjected to heavy stresses should always be annealed for two to four hours at a temperature above the upper critical point (Art. 697). Annealing not only removes internal shrinkage stresses but also refines the grain and renders the casting stronger, tougher and more ductile. The toughness of castings of uniform section can be considerably increased by quenching in oil and reheating to a temperature just below the critical range.

**616. Statistics.**—Production statistics of standard shapes and sections are shown in Table 1.

TABLE 1.—TONNAGE OF PRINCIPAL IRON AND STEEL PRODUCTS OF UNITED STATES FOR 1914. (LONG TONS)

*From Mineral Industry, 1914*

	Iron.	Steel.	Total.
Rails.....		1,945,095	1,945,095
Plates and sheets.....	56,590	4,662,656	4,719,246
Nail and spike plates.....	4,725	33,848	38,573
Wire rods.....	731	2,430,983	2,431,714
Structural shapes.....	1,981	2,029,143	2,031,124
Merchant bars.....	563,171	1,960,460	2,523,631
Reinforcing bars.....		288,471	288,471
Skelp, flue and pipe iron.....	264,340	1,718,091	1,982,431
Long angle splice and tie-plate bars, etc.....	50,295	372,757	423,052
Hoops.....		211,028	211,028
Bands and cotton ties.....	180	345,739	345,919
Rolled sheet piling.....		35,314	35,314
Railroad ties.....		33,249	33,249
All other finished rolled product.....	223,802	714,116	937,918
Forging blooms, billets, etc.....	500	331,024	331,524
Exports of blooms, billets, sheet bars, etc.....	1,461	90,446	91,907
	1,167,776	17,202,420	18,370,196

## CHAPTER XX

### FORMATION AND STRUCTURE OF ALLOYS

#### ALLOYS IN GENERAL

**617. Reasons for Making Alloys.**—In general, the properties desired in a metal to be used in engineering construction are not embodied to the best advantage in any single metal, and recourse is had to the mixing of two or more metals to attain the desired end. Such mixtures of metals or metallic substances are classed as alloys, and they form one of the most important subdivisions of metallurgy.

**618. Mixtures.**—The properties of alloys are influenced by the nature and proportions of the components; but of equal importance, in fact in many instances of predominating importance, is the influence of the character of the mixture. The constituents may form a simple mixture, coherent of course, but existing in the mass as distinct individuals. The properties of such a mixture are in large measure an average of those of the components, and vary with the relative proportions; this general relation may be modified to varying extents by the relative adhesion of unlike particles as compared with the cohesion of the like constituents, and by the degree of fineness of the aggregate as affected by the nature of the components and physical conditions of manufacture.

**619. Chemical Compounds.**—In extreme contrast to the above, the components may have such a degree of affinity that they unite in atomic proportions to form a chemical compound, a new unit substance in which the individuality of the constituents is lost, and which may have physical properties distinct from and unrelated to those of the components of the alloy. Intermetallic compounds play an important role in the consideration of alloys, the carbide  $\text{Fe-C}$  being of especial influence on the properties of steel.

**620. Solid Solutions.**—The above-mentioned conditions are largely special, and represent the extremes or end relations in the possible degrees of miscibility of the constituents of alloys. Between is the wide gap in which the components are mutually soluble to the extent that they become blended into a homogeneous unit with loss of visible evidence (even microscopic) of isolation of particles, and yet where there is not that absolute loss of individuality which accompanies chemical union in

atomic proportions. Solid solutions are important factors in the study of alloys; the resultant properties may differ in varying degrees from those of the components, according as the order of solubility approaches that of definite chemical union as a new compound.

Constituents of an alloy may be only partially soluble; that is, solid solutions may result from addition of either component to the other up to certain limits of saturation, beyond which there results a simple mixture between these saturated solid solutions. Again, a nintermetallic compound may form a simple mixture or a solid solution with the components of the mixture, or with a second compound of different atomic proportions in the same series of constituents.

**621. Methods of Making Alloys.**—Alloys may be made in various ways; those of greatest importance in the manufacture of materials for engineering construction are by fusion of the constituents and solidification after mixture, and by diffusion, where the body metal is in the solid state, and the diffusing material is solid, liquid or gas. The first-named method is most common and is employed where uniformity of material is desired throughout the entire section; while the latter is chiefly of value in imparting a surface condition to a section which will give it a desired composition and properties different from those of the body material proper. In the formation of alloys by fusion, complete solubility of the constituents in the liquid state is usually desired (that is, within the useful range of composition); otherwise differences of specific gravity of the components will result in liquation and consequent irregularities of composition throughout the mass, especially if the mixture is held in liquid and quiescent condition for a period of time. But the solubility of metals is a function of temperature, pressure, and particularly of the state of the constituents. Thus, while there may be perfect miscibility while fusion exists, solidification may result in a solubility which is complete, partial, or nil. Perfect homogeneity of liquid is no criterion of structure in the solid; the latter may be homogeneous or heterogeneous—a solid solution or a simple mechanical mixture. Furthermore, the degree of miscibility, or especially the limits of saturation, being a function of the temperature, may alter by diffusion with changes of temperature below that of solidification. Diffusion in the solid state is, however, comparatively slow, and it is only by the maintenance of relatively high temperatures for long periods of time that alterations of structure occur. Fortunately such variations are not the usual accompaniment of the changes in temperature to which constructive materials are subjected in customary service.

**622. Allotropy.**—Certain metals have characteristic changes of state (evidenced by alteration of crystal form, physical properties, etc.) while in one of the usually recognized states of solid, liquid or gas. Such a



modification is called "allotropic." A solid alloy, which has as one of its constituents a metal exhibiting allotropy, may undergo transition in passing through the temperature normal for such allotropic change. Iron undergoes allotropic modifications in the solid state, and these are of extreme importance in heat-treatment processes.

Variations in degree of solubility of constituents result in differences in type of structure of the alloy; these in turn are accompanied by variations in its physical properties. Utilization of the effects of alloying vastly broadens the field of usefulness of the primary metals. The range of utility is influenced and complicated by the changes accompanying variations in character and number of constituents, their miscibility relations and the modifying effects of allotropy.

**623. Crystalline Structure of Metals.**—Metals are crystalline, that is, they are made up of an aggregate of crystals of definite form which is a constant for each metal. Solidification from a melt, in the case of a single, pure metal, will be by formation of numerous small definitely formed crystals in those parts of the melt which have reached the freezing temperature, which is a constant for the single metal. Upon these crystals as nuclei, others will build as solidification progresses, attaching themselves regularly with crystal faces together, and making up aggregates, each of which has a resultant orientation of axes of crystallization dependent upon the purely accidental position of its nucleus in the melt. As solidification approaches completion, there will be interference to further growth along lines of contact between different aggregates, and with complete solidification, a single pure metal will consist of a large number of grains, with irregular boundaries resulting from the contact lines of the individual crystalline aggregates which constitute the several grains. Typical structures of this character are given in Fig. 1. The structure is of necessity homogeneous, and the strength and other properties will be those of the single metal; however, variations may result from differences in average size of grains, which in turn may be influenced by cooling conditions which affect the number and distribution of nuclei of crystallization. Variations in properties may also be caused by differences in the relative cohesion along the contact faces of the crystals making up the individual grains, as compared with that along the boundaries or contacts between the separate aggregates or grains.

**624. Effects of Solubility Relations in Alloys.**—When two or more metals are mixed to form an alloy, solubility relations have great influence upon the progression of freezing and the structure of the resultant alloy. If the relations of the constituents are such that chemical combination results, and the amounts are of proper atomic proportions, solidification will progress in a manner entirely similar to that of a single pure metal. An intermetallic compound is essentially a unit substance; it will freeze

at constant temperature, and it must exhibit a homogeneous structure consisting of a mass of grains, each composed of an aggregate of uniform crystals.

With relations of solubility other than that of chemical combination, solidification proceeds selectively, and is almost always spread over a range of temperature, as distinguished from the constant freezing temperature of a single metal. In subjecting a material to conditions which tend to change its state, there is a resistance opposed by the substance which tends to counteract such transition. In the case of a single substance, this is by sacrifice of part of the mass, which in freezing liberates its latent heat of solidification and thus by a tendency to hold up the normally falling

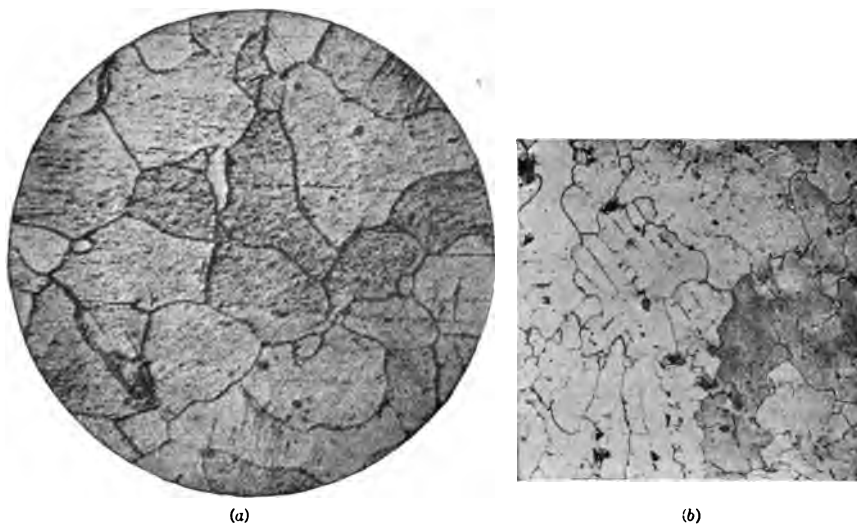


FIG. 1.—Photomicrographs of Pure Metals Showing Crystalline Structure (a) of Swedish Iron (b) of Copper. (Fig. 1b.—Courtesy of Sauveur and Boylston.)

temperature, tends to keep the remainder of the mass in a liquid condition. With two or more metals in mixture, there is the possibility of resistance to change of state by liberation of heat during freezing of a portion of the mass, as noted above for a single metal; in addition there is the added possibility of sacrifice of some portion of the mass, which differs in composition from the body material in the direction of a higher temperature of freezing. Thus there is enrichment, by such selective freezing, of the remaining portion of the melt in the direction of a composition of lower temperature of solidification, and a resistance thereby to the external conditions tending to promote solidification. By continued and progressive selection, the freezing of alloys may be, in fact very generally is, spread over a range of temperature, and differs in this respect from the solidification of a single pure metal, which must occur at constant temperature.

The structure of the alloy will be influenced by its method of freezing and the solubility relations; the former is, however, a function of the latter, and thus relations of solubility of the solidified constituents of the alloy are governing factors in the final structure. Assuming complete solubility in the solid, freezing will proceed by progressive selection, and will be spread over a range of temperature, as noted above. However, the assumed condition of solid solution implies that initial heterogeneity because of progressive selection, should be effaced by diffusion, and the resultant alloy should, therefore, exhibit a homogeneity of structure of the granular type characteristic of the single pure metal or the intermetallic compounds. But interdiffusion of solid particles is likely to be relatively slow; thus by rapid cooling of the alloy through and below the freezing range, opportunity for such blending to a homogeneous mass may be checked to a greater or lesser degree, and a heterogeneity varying throughout the individual grains from the composition of the first frozen particle to that of the last, may result in the solid alloy.

With solubility nil between the constituents of an alloy, resulting structure must be heterogeneous. Freezing from the melt will be by selection, with solidification of that one of the constituents which will, by its elimination, progressively enrich the melt in composition towards that of lowest freezing-point. This first portion to freeze will have an opportunity to solidify with considerable definiteness and regularity with respect to its crystal form, and without interference between the individuals, since there is freedom for movement in the remaining liquid. This liquid is finally forced to composition of lowest and final freezing temperature—the so-called eutectic temperature and composition, and constant for a particular alloy—when it will solidify and occupy such space as may remain between the particles making up the portion frozen during selection. Again, however, because of lack of solubility between the solid constituents, the eutectic portion must separate into its individuals on solidification and in itself must be heterogeneous.

**625. Evidence of Alloy Constitution Furnished by Thermal Measurements.**—Since variations in alloying relations are accompanied by changes in physical properties of the alloy, the latter may form the basis of interpretation of the type of miscibility of the components. Two methods have, however, come into prominence as of particular value in the study of the constitution of alloys—thermal measurement and microscopic examination. The former is based upon the principle that changes in state of a substance are accompanied by change in its internal energy content, which is manifested by an absorption or liberation of heat. A body without transitions in the temperature range will absorb heat from surroundings at higher temperature, or radiate heat to surroundings at lower temperature, with perfect regularity, and a graphical representation

plotted as a function of temperature and time, will show an approximately logarithmic curve without breaks from smoothness or regularity. In unit time, cooling will be very rapid when the body is at a high temperature compared with the surroundings; it will become less and less rapid as the body temperature more nearly approaches that of the surroundings. Should there be a change of state within the temperature range of investigation, the accompanying absorption or liberation of heat will cause a deviation from the normal law of heating or cooling, and such transition will be manifested by a jog or break from the normal curve extending over a time interval equal to that of heat absorption or liberation due to the transition. A familiar example of the above is the arrest of temperature during melting or freezing of a single pure substance.

By inserting a sensitive pyrometer into a molten alloy of any desired composition, and noting the temperature-time variations during normal cooling and solidification to atmospheric temperature, a "cooling curve" may be plotted which will be a record of the internal condition of the particular alloy within the temperature range employed. Changes of state at solidification or because of allotropy will be indicated by breaks from a smooth curve for intervals of temperature and of time corresponding to such transitions.

**626. Cooling Curves.**—Typical cooling curves are given in Fig. 2a to 2l. Fig. 2a shows the smooth curve of a substance cooling without

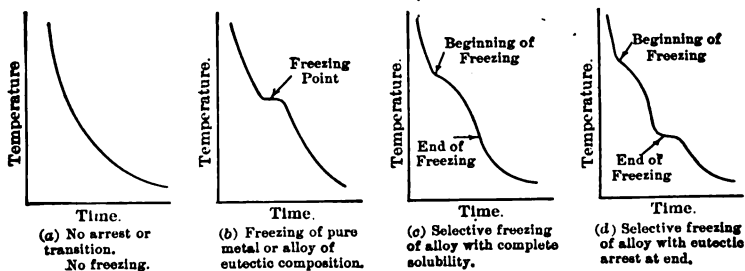


FIG. 2.—Typical Cooling Curves.

transition, plotted as a function of temperature and time. The freezing of a single pure metal, an intermetallic compound, or the eutectic portion of an alloy which solidifies at constant temperature, introduces the arrest illustrated in Fig. 2b, over a time interval corresponding to the liberation of the heat of solidification. In an alloy where the freezing is selective, the progressive shift of composition distributes the process over a range of temperature. For complete solubility in the solid, shown in Fig. 2c, the progressive solidification of particles which are solid solutions of two or more constituents, and the blending of these into a homogeneous unit as freezing proceeds, results in gradual approach of the break in the

cooling curve to the normal with progressive elimination of the liquid portion. Where solubility is nil between the solid constituents, selective freezing introduces a break similar to that of the solid solution type while such selection proceeds. But since the solid portion now consists of one pure substance alone, the shift in composition of the liquid is to a portion—which may be the other constituent of a two-component mixture, but is usually some fixed, intermediate mixture of the two—which solidifies at constant temperature. The form of cooling curve is shown in Fig. 2*d*. For the particular metal under investigation, a cooling curve which exhibits freezing over a range of temperature without arrest at constant temperature denotes complete solubility in the solid; where freezing is spread over a range of temperature with final solidification at a constant temperature, solubility is nil between the solid constituents, or the composition is beyond the saturation limits of an alloy of partial miscibility; with constant freezing only, the metal is a single pure substance, an intermetallic compound, or the eutectic mixture of an alloy with solubility nil or partial between the two components.

Cooling curve records may be obtained for progressive variations in composition of any alloy series. The temperatures of transition, or “critical points,” particularly the temperature points on this temperature-composition diagram, will give a graphical record of the internal conditions of the alloy series during the cooling and solidification interval. These “solidification,” “constitution,” “freezing-point,” or “equilibrium” diagrams assume types which are characteristic of the solubility relations of the components.

In a two-component mixture of metals (or binary alloy) a solubility which is complete while the constituents are in the liquid state may after solidification remain perfect, or may become partial or nil.

#### BINARY ALLOYS WITH LIQUID SOLUBILITY PERFECT, SOLID SOLUBILITY NIL

**627. Significance of the Freezing-point Diagram.**—A typical freezing-point diagram is given in Fig. 3. Composition of the two components, *W* (white) and *B* (black), is indicated on the horizontal axis, and temperature on the vertical axis. The lines *CED*, and *FEG*, are the loci of transition points obtained from cooling curves of individual alloys of the series, and indicate respectively the commencement and completion of solidification. That solubility is nil in the solidified alloy, is indicated by the fact that the line *FEG*, making the completion of solidification, extends completely across the diagram. Thus, whatever may be the composition of the alloy chosen in the series *W*–*B*, completion of solidification will always be at a constant temperature *F*–*G*; and this, in turn based upon

interpretation of experiment, and particularly upon the deductions of the "Phase Rule," is evidence of a lack of solid solubility between the components *W* and *B*.

The solidification temperatures of *W* and *B* are at *C* and *D* respectively. A feature of interest is that upon addition of *W* to *B*, or *B* to *W*, freezing begins at progressively lower temperatures with each addition of the second element, until finally a composition *H* is reached at the intersection of *CE* and *ED*, which has the lowest freezing temperature of any alloy of the series, and one which is usually below that of either of the constituent metals. This marks the so-called "eutectic point" of composition and temperature, which is constant for the particular alloy series, but need not be, in fact usually is not, at a composition of definite atomic proportions of the components.

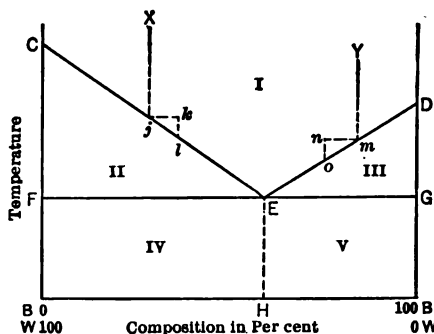


FIG. 3.

The solubility diagram is divided into five characteristic areas: I, the region of the melt, a liquid solution of *W* and *B*; II, and III the freezing range, during which the alloy consists of a partially solid, partially liquid mass; IV and V, the regions of the completely solidified alloy.

**628. Behavior of a Given Alloy in Freezing.**—Any alloy of composition and temperature *X*, is a homogenous liquid, and will remain so until the temperature has dropped to *j*. At this point freezing must begin, with change in solubility relations from complete to nil during the change in state from liquid to solid. That affinity between *W* and *B* which delayed freezing for a time, is lost or minimized after solidification. But by selection during freezing, sacrifice of a portion of the mass will retard solidification of the remainder of the melt. At temperature *j*, a solid particle of *W* will separate from the melt; and by this depletion in constituent *W*, the melt is enriched in the direction *k*, and will not further solidify until the temperature falls to *l*. At such time, further sacrifice of component *W* by solidification, will again result in *B*-enrichment of the melt and further fall of temperature to effect additional solidification. This step by step shift of composition and temperature, with progressive solidification of *W*, will follow the line *CE* until *E* is reached. Since these solid particles of *W* are forming in a melt and their primary crystallization is completed while there is still some liquid remaining, these primary aggregates of *W* should exhibit a regularity of outline which is in the accord with the normal habit of crystallization of the metal *W*.

Selective freezing having forced the remaining liquid portion of the original alloy to the eutectic composition and temperature  $E$ , it must solidify in its entirety at this temperature since no further selection during freezing, either by sacrifice of  $W$  or of  $B$ , will withhold solidification until the temperature falls to a lower point. The eutectic will freeze at constant temperature, in a manner similar to a single metal; but, since our alloy constituents are immiscible in the solid state, the homogeneous liquid particle must split up during solidification into two distinct particles of  $W$  and  $B$ . The degree of aggregation of the composite eutectic will depend upon cooling conditions, which influence the rapidity of solidification and of separation of  $W$  and  $B$ , and the possibility of coalescence of contiguous  $W$  and  $B$  particles, like to like. The eutectic structure will be a composite mixture of finely or coarsely granular or lamellar character; it must be heterogeneous and occupy the space existing between the relatively coarser and more regularly formed aggregates of  $W$  resulting from primary crystallization.

#### 629. Structures of Alloys of Perfect Solubility—Solid Solubility Nil.—

An alloy structure of definite type will accompany the method of solidification outlined above. Assuming  $W$  to crystallize as cubes, such portion of this constituent as solidifies during the period of primary selective freezing, will make up a ground mass of white cubes, being free to build up on the cubical nuclei of initial crystallization because of the relative mobility of the remaining liquid portion during the temperature range of selective crystallization. The eutectic, on the other hand, will occupy the space remaining after the interval of primary crystallization, but will exist in

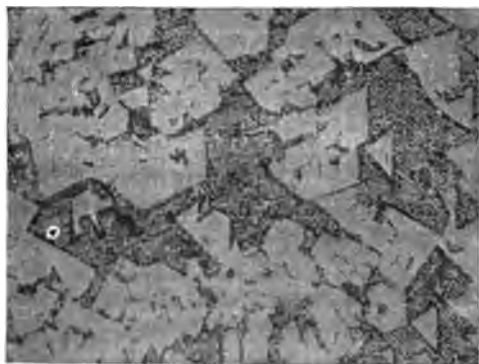


FIG. 4.—Photomicrograph of an Alloy of Equal Parts of Lead ( $B$ ) and Antimony ( $W$ ), showing cubes of excess antimony in a eutectic of lead and antimony.

this area as a composite granular or lamellar structure of white and black particles. For all alloys on the  $W$  side of the eutectic proportions,  $W$  will be the excess substance, exhibiting primary crystallization; while all of the  $B$  constituent will appear in the eutectic. The relative proportion of primary constituent  $W$  and eutectic will depend upon the initial composition of the alloy; the more rich it is in  $W$ , the less will be the relative quantity of eutectic.

A typical structure is shown in Fig. 4. The white cubes of  $W$ , exhibiting various polygonal

shapes depending upon the purely chance form of orientation and distribution of the cubes, and the plane of cutting of the section, are imbedded in a granular white and black eutectic of relatively finer state of aggregation of both *W* and *B*.

For alloy compositions on the *B* side of the eutectic as at *Y* in Fig. 3, freezing will be as rigidly selective as that outlined above in the discussion of those where *W* is the excess metal (as related to the eutectic proportions). However, on reaching the temperature *m*, *B* will be the first to crystallize, since by a removal of some of this constituent from the melt, the latter will be enriched in *W* in the direction *n*, and will therefore not solidify further until the temperature has fallen to *o*. At this time, additional sacrifice of *B* by solidification will repeat the composition-temperature shift; the selective crystallization will proceed until the eutectic point is reached, when final solidification, accompanied by separation into a composite mixture, will result.

All alloys on the *B* side of the eutectic proportions (*B* is in this case called the excess substance, regardless of the particular weight relations)

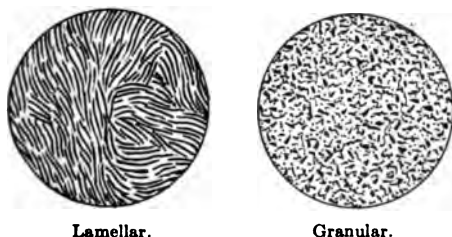


FIG. 6.—Lamellar and Granular Types of Eutectic Structure.

Should the initial alloy be of eutectic composition, there will be no primary crystallization of either *W* or *B*. Upon reaching the eutectic temperature, freezing will proceed at constant temperature, with separation of the constituents into the composite black and white structure characteristic of the eutectic. Typical structures of granular and lamellar types are shown in Figs. 6 and 7.

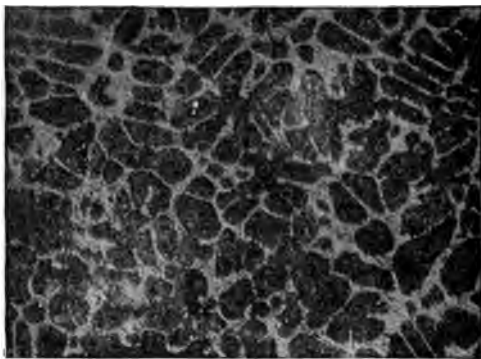


FIG. 5.—Photomicrograph of an Alloy of Antimony (*W*) and Lead (*B*) Containing 95 per cent Lead. Note the oval black areas of lead surrounded by the eutectic network.

will exhibit a structure of black aggregates of regular outlines due to primary crystallization, together with a composite black and white granular or lamellar eutectic, the latter similar in all respects to that characteristic of the *W*-rich alloys described previously. The typical structure is shown in Fig. 5, assuming *B* to form aggregates of oval form.



**630. Summary for Alloys with Solid Solubility Nil.**—To summarize, binary alloys of the type where solubility is complete in the liquid state, and nil in the solid, are characterized by the presence of a eutectic throughout the series. All intermediate alloys, therefore, solidify at temperatures lower than those of the two constituent metals, except for the unusual limiting case where the eutectic point is at the end of the series coincident with the constituent metal of lowest freezing temperature. Freezing is spread over a range of temperature and is rigidly selective throughout the series, with primary crystallization of that component which is in excess of the eutectic proportions. The eutectic occupies

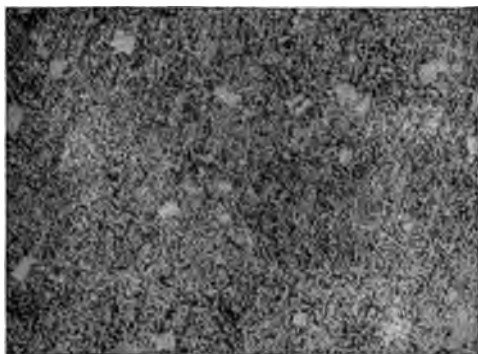


FIG. 7.—Photomicrograph Showing Eutectic Alloy of 13 Per Cent Antimony and 87 Per Cent Lead with a Few Segregated Antimony Crystals.

the areas remaining after primary crystallization, and its quantity is inversely proportional to the distance of the particular alloy from the eutectic composition. The eutectic always solidifies at constant temperature, is always of constant proportions, which are only coincidentally of atomic ratios, and is a composite of relatively finer state of aggregation than the excess metal of primary crystallization. All alloys of the series are heterogeneous throughout, and consist of an excess substance, which may be either pure metal, plus the eutectic. Typical alloys of this series are lead-antimony and lead-tin.

#### BINARY ALLOYS WITH LIQUID SOLUBILITY PERFECT, SOLID SOLUBILITY PERFECT

**631. Typical Freezing-point Diagram.**—The assumption of a solubility in the solid state, which is complete throughout the entire alloy series, implies of necessity perfect homogeneity of structure. Experimental evidence, however, indicates selectiveness during freezing, with consequent heterogeneity which is effaced only after solidification.

A typical freezing-point diagram is given in Fig. 8, plotted in the customary manner as a function of temperatures for ordinates against composition abscissæ for two components *W* (white) and *B* (black). The loci for commencement and completion of solidification, as determined by transition points, noted on cooling curves taken on various composi-

tions within the series, are given by the lines  $CnD$  and  $CoD$ , respectively. Between these two curves, indicated by the area II, is the region of selective freezing, spread over a range of temperature for any mixture of the series. Areas I and III are, respectively, the regions of the completely molten and completely solid mixtures.

**632. Behavior of a Typical Alloy in Freezing.**—For an alloy of composition  $X$ , freezing will begin when the temperature falls to  $e$ . Solidification of a particle richer in  $W$  than the alloy  $X$ , will enrich the remaining liquid in  $B$ , or in the direction  $k$ , and thus necessitate further lowering of temperature before freezing will progress. Since the solid particles must be at the same temperature as the melt, equilibrium conditions being assumed, the composition of this first frozen solid will be  $f$ . Since there is complete solubility in the solid state, this particle will be a homogeneous blend, a solid solution of  $W$  and  $B$ , which has no tendency to split up into the constituent metals.

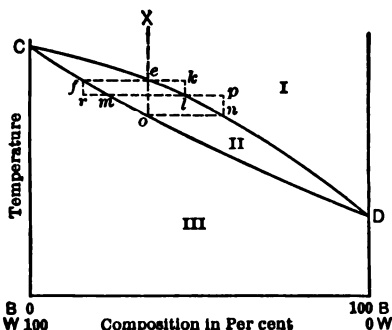


FIG. 8.

When the temperature falls to  $l$ , a second particle will crystallize from the melt, again enriching the latter in  $B$ , and prolonging the life of the liquid by the composition shift to the right. Meanwhile, however, the temperature of the first solid particle has fallen to  $r$ . Because of solubility in the solid,  $r$  and  $m$  will tend to diffuse to a homogeneous unit; and furthermore, to draw from the liquid sufficient of  $B$  to enrich the resultant blended solid to a composition  $m$ .

The above described step by step enrichment of the liquid by selective crystallization of the solid, followed by necessity of lowering of temperature to ensure further solidification, will proceed until the whole of the alloy is solid. If the diffusion of the solid particles has kept pace with their crystallization from the melt, solidification will be completed when the last liquid portion reaches a composition  $n$ . The last frozen particle will have a composition  $o$ ; this corresponds to the previously solidified aggregate resulting from diffusion and enrichment in such  $B$  as required, by withdrawal from the melt. Thus the resultant alloy, provided diffusion is complete, will be a homogeneous light gray solid of the composition  $o$ , identical with that of the original melt  $X$ .

Should diffusion in the solid fail to keep pace with the rate of crystallization, freezing will not be completed until the melt has reached a composition between  $n$  and  $D$ ; the last frozen particle will in this case have

a composition between *o* and *D*. And because of lack of diffusion of the solid particles, the resultant alloy will exhibit a heterogeneity in the individual grains, progressively shading in color from the lighter gray of a nucleus of *W*-rich material of composition somewhere between *f* and *o*, to a relatively darker gray of the last freezing portions of *B*-rich material of composition between *o* and *D*. The extreme or limiting condition, provided no diffusion in the solid resulted, would be a nucleus corresponding to the first frozen material *f*, shading progressively to *B*, marking the boundaries of the individual grains. But in all cases, the structure should exhibit a mass of grains, similar to that of a pure metal (Fig. 1); with each grain a crystalline aggregate having the homogeneity of a solid solution under ideal conditions, but actually approaching this only to the degree that diffusion in the solid has effaced the heterogeneity resulting from selective freezing.

Attainment of homogeneity depends upon the characteristics of the constituent metals, and upon the condition of cooling. Since diffusion in the solid is relatively much slower than in the liquid state, slow cooling at temperatures just below the solidification range is rather an essential condition to promote homogeneity; rapid solidification and cooling to normal temperatures, on the contrary, promote heterogeneity of the characteristic shading-off type discussed above.

**633. Summary for Alloys of Perfect Solid Solubility.**—To summarize, binary alloys of the type having complete solubility in both liquid and solid states, solidify by a process of selective freezing. This is not rigid, however, and the heterogeneity of structure resulting from such selection, is effaced by diffusion, under ideal conditions of cooling; the final alloy is then a solid solution with homogeneity of structure. The so-called "onion peel" type of structure may result if cooling conditions are such that there is not complete effacement of the heterogeneity of selective crystallization. All alloys of the series must freeze over a range of temperature, and there can be no solidification at constant temperature corresponding to the eutectic of alloys of the immiscible type. Typical alloys of this type are: iron-manganese and antimony-bismuth.

#### BINARY ALLOYS WITH LIQUID SOLUBILITY COMPLETE, SOLID SOLUBILITY PARTIAL

**634. Typical Freezing-point Diagram.**—Partial solubility in the solid state implies perfect miscibility, with accompanying homogeneity of structure, at either or both ends of the series, up to particular limits of saturation. Beyond these limits, there must be separation of the constituents, and heterogeneity of structure. The relations are a combination of the two types previously discussed.

A typical freezing-point diagram is given in Fig. 9. There are seven characteristic areas: I, the molten solution; II and III, the regions of selective crystallization; IV and V, the regions of solid solubility; VI and VII, the regions beyond the saturation limits, with heterogeneity in the solid alloys. The limits of solid solubility are marked by the eutectic line *GEF*; that is, the presence of a eutectic is evidence of final solidification at constant temperature and composition, and of consequent separation, in accord with the deductions of the "Phase Rule," into the distinct constituents.

**635. Behavior of a Typical Alloy in Freezing.**—An alloy of composition *U* will solidify entirely in accord with the method outlined for complete solubility. There will be selectiveness in freezing, and solidification will be spread over a range of temperature from *k* to *m*, because of a progressive shift in liquid composition from *k* to *n*. But the heterogeneity of structure resulting from the selective crystallization will be effaced by diffusion of the solid particles, under proper conditions of cooling, so that the final alloy should be a heterogeneous solid solution of composition *m*, and of light gray color due to the preponderance of white constituent *W*. This type of crystallization and resultant homogeneity of final structure will be characteristic of all alloys within the limit of solubility *WH*.

The above reasoning will hold, likewise, for the solidification of an alloy of composition *Z*. The solution will, however, be by progressive crystallization of *B*-rich particles, and consequent enrichment of the remaining liquid in *W*, thus prolonging the temperature range. Solid alloy *Z* will differ from *U* only in its darker color due to higher content of the black constituent *B*. At this end of the series, the saturation limit is *WJ* per cent of *W*, added to *BJ* per cent of *B*.

An alloy *V* will begin to freeze at temperature *o*, with separation of first frozen particle *p*. Its removal from the melt will enrich the melt in *B*, thus necessitating lowering of temperature to ensure further solidification. Selective crystallization, accompanied by progressive shifts of composition and temperature, will take place until the temperature-composition *E* is reached. Meanwhile, diffusion of the solid particles, together with extraction of sufficient *B* from the melt to supply the deficit, will result in a homogeneous solid aggregate of composition *G*. However, since *V* lies to the right of *G*, all of the molten liquid has not as yet been used up. Again, the limit of solubility of the solid has been reached,

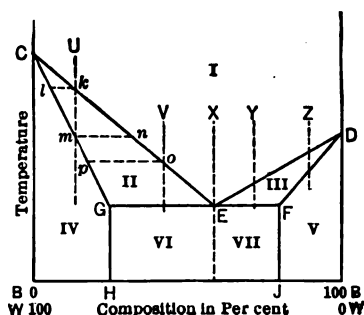


FIG. 9.

as well as the eutectic temperature of freezing. Because of the latter, no further selection will enable solidification to be deferred to a lower temperature. Final solidification must now proceed at constant temperature, accompanied by immediate separation of the solid constituents of the eutectic. Due to partial solid solubility, the eutectic will not be a composite of each of the two pure components *W* and *B*, but will consist of a heterogeneous mixture of particles of *W* saturated with *B*, and *B* saturated with *W*; in other words, of a granular or lamellar composite of compositions *G* and *F*.

The final structure will therefore consist of a primary aggregate of composition *G*—a solid solution exhibiting its particular definiteness of crystallinity—together with a eutectic occupying the remaining area, and composed of a composite mixture of particles of composition *G* and *F*, each a saturated solid solution. The structure is heterogeneous, and is characteristic of all alloys between *G* and *E*; the differences between these alloys being in the quantitative proportions of primary substance with respect to eutectic, the amount of the latter increasing from 0 to 100 per cent as the composition of the particular alloy varies from *G* to *E*.

Alloys of composition between *E* and *F* solidify in a manner similar to those just discussed. The excess substance of primary crystallization is, however, a saturated solid solution of composition *F*, which will structurally exhibit its particular definiteness of crystalline habit. The eutectic will be identical in composition with that resulting in alloys from *G* to *E*; that is, a heterogeneous composite of two saturated solid solutions *G* and *F*. The quantity of eutectic will vary from 0 to 100 per cent as the composition of the original alloy is shifted from *F* to *E*.

Should the original alloy be of eutectic proportions *E*, solidification will not begin until the temperature has dropped to *E*. It will then proceed at this constant temperature, with accompanying separation of the two constituents *G* and *F* to form the characteristic granular or lamellar composite, making up the entire structure.

**636. Summary for Alloys with Solid Solubility Partial.**—To summarize, binary alloys of the type where solubility is complete in the liquid state, and partial in the solid, are characterized by a eutectic for part of the series, and this eutectic marks the limits of solubility in the solid. Freezing is selective throughout the series. It is not rigidly so within the limits of saturation, and the heterogeneity of selective crystallization should be effaced by diffusion in the solid, and homogeneous alloys should result. Beyond the limits of solubility, the heterogeneity cannot be effaced by diffusion, and the resultant alloys consist of an excess substance of primary crystallization, plus a eutectic. The excess substance is a saturated solid solution, of either component of the series saturated with the other, depending upon the composition of the particular alloy with res

to that of the eutectic. The eutectic is a composite of the two components of the alloy series, each saturated with the other, and is always of the same composition for the particular series. All alloys of compositions within the limits of solubility are structurally homogeneous, provided cooling conditions have enabled diffusion to complete itself. Beyond the limits of solubility, the structure is heterogeneous.

Modifications in the form of solidification diagram for alloys of this type of partial solubility, are given in Fig. 10. The differences involved are only in details of progress of solidification, and not in the character of structure resulting. The reader is referred to treatises on metallography for detailed discussion of these modified forms.

Typical alloys of this class are those of aluminum and zinc.

## ALLOYS OF MORE THAN TWO COMPONENTS

### 637. Complexity Due to Metallic Compounds.—

Additional complexity is introduced into the study of alloys by the presence of more than two components in the mixture and the possibility of formation of chemical compounds between the constituents.

If an intermetallic compound, for example, of the atomic proportions  $W_1B_1$  forms in the alloy series  $W-B$ , this compound is a homogeneous, unit substance, and usually is

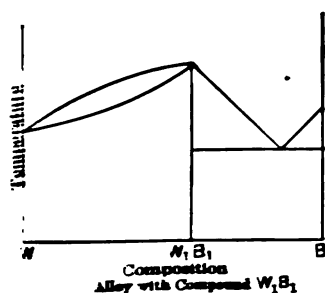


FIG. 11.

unlike the constituent metals in its melting-point or general physical properties. It enters into alloying relations with  $W$  on the one hand, and  $B$  on the other. Virtually, therefore,  $W$  and  $B$  no longer have simple relations with one another; the series becomes a duplex one of  $W$  with  $W_1B_1$  and  $W_1B_1$  with  $B$ . There is a duplex set of solubility relations, each independent of the other. Fig. 11 illustrates such a series, in which  $W_1B_1$  and  $W$  have a completely miscible relation in the solid state, whereas  $W_1B_1$  and  $B$  form heterogeneous alloys because of lack of solubility in the solid. No alloys of the series will have free  $W$  and free  $B$  together. A second compound  $W_2B_2$  would make the couples of the complete series  $(W-W_1B_1)$ ,  $(W_1B_1-W_2B_2)$ ,

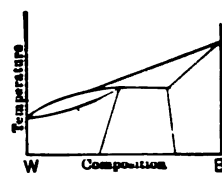
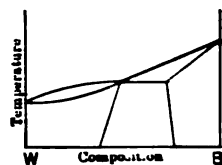
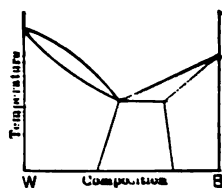


FIG. 10.

( $W_2B_2-B$ ), each in reality a binary series independent with respect to the others. The simplest method of treatment of a binary alloy with intermediate chemical compounds is to break the series into its component groups, and to consider each as a simple binary alloy having one of the three fundamental types of solubility relations discussed heretofore.

**638. Difficulties Encountered with Several Components.**—The consideration of alloys with multiplicity of components becomes increasingly difficult with increasing number of constituent metals. The possible groups of binary, ternary, etc., relations becomes very great, and it is an extremely arduous task to obtain all of the cooling curves needed to make up the complete constitution diagrams. The possible formation of complex chemical compounds also adds to the difficulty. Again, the graphical representation of the data becomes complex, or even impossible. For a ternary series, it is possible to represent the composition relations by means of triangular coordinates; and with temperatures plotted on axes perpendicular to this triangular plane, the solubility diagram takes the form of a space model in three dimensions, as compared with the two-dimension, plane diagrams of the binary alloys.

Much experimental data has been obtained for the relations in binary alloys; on the contrary, but little has been done for greater multiplicity of components. At the present time the most satisfactory method of treatment of the different industrial alloys, most of which have several component metals, is to consider first the binary relationships of the two important constituents, and subsequently deal with the modifying influences of the various secondary elements.

## CHAPTER XXI

### CONSTITUTION OF IRON AND STEEL

**639. Necessity for Alloying Pure Iron.**—Very little of the iron which is employed in engineering construction is pure; in fact, by far the greater proportion is purposely alloyed with one or more elements in order that the modification of the inherent properties of the iron will enhance the utility of the resultant metal.

**640. Classifying Iron and Steel.**—Dating from the time when all iron products were the result of primitive methods of manufacture, they were classed as (a) iron, material which would not harden appreciably on sudden cooling; and (b) steel, material which did harden by such treatment. Later, the nonforgeable cast iron came into the fold as a third variety of product. The above classification seemed to meet all requirements until the advent of modern processes of steel manufacture. With the development of Bessemer and open-hearth methods, it became possible to make a product of such low carbon content that it did not harden any more than the older irons on quenching from a red heat. Yet there was a greater selling value attached to the name "steel"; there was the fact that the low-carbon material resulted from the modification of the merest details of manipulation of processes which equally readily made high-carbon product, or "steel"; and there was no sharp dividing line between the classes of material. Thus, the name "steel" became associated with the product of modern processes of manufacture; and with their present day predominance it has become rather firmly rooted in our nomenclature as covering all articles made by Bessemer, or open-hearth methods, regardless of their inability to harden on sudden cooling.

The current usage in this country, as well as in England and France, is to classify as

- (1) Wrought iron—the product of the puddling furnace or sinking fire.
- (2) Steel—the product of the cementation process, or malleable compounds of iron made in the crucible, converter, or open-hearth furnace.

In Germany the effort has been to group the products more in accordance with their physical properties, and the soft steel of the converter and open-hearth processes is called "ingot iron."



The definitions which follow were proposed by a Committee on Nomenclature to the Sixth Congress of the International Association for Testing Materials in 1912.\*

#### DEFINITIONS PROPOSED FOR THE PRINCIPAL FERROUS METALS

**641. Cast Iron.**—Definition. Iron containing so much carbon that it is not usefully malleable at any temperature.

Remarks. In America, besides the foregoing generic meaning, "cast iron" is used also in a specific sense which excludes "pig iron" (see below) and is restricted to cast iron in the form of castings other than pigs and to remelted cast iron suitable for such castings. The cast iron of commerce is reduced from the ore, usually in the iron blast furnace, in direct contact with solid carbon, and is then tapped from the furnace in a molten state. It always contains an important percentage of carbon, usually from 2.5 per cent to 4.5 per cent, and in most cases an important percentage of silicon.

There are three chief varieties of cast iron.

*Gray cast iron:* Relatively soft, and characterized by the presence of sheetlets of graphite, often forming an irregular skeleton. This is the variety used chiefly for engineering work

*White cast iron:* Extremely hard and brittle, characterized by having all or nearly all its carbon in the combined state, and by its consequent lack of graphite.

*Mottled cast iron:* Intermediate between gray and white cast iron.

**642. Pig Iron.**—Cast iron which has been cast into pigs direct from the blast furnace or its equivalent. This name is also applied to molten cast iron which is about to be so cast into pigs, or is in a condition in which it could readily be cast into pigs.

**643. Mixer Metal.**—Molten cast iron which has been passed into or through a metal mixer.

**Iron Castings.**—Definition. Cast-iron castings.

Remarks. They are usually made either (1) by remelting cast iron in a cupola or other furnace, or in crucibles; or (2) from molten iron direct from the blast furnace.

**644. Malleable-iron Castings.**—Definition. Iron which is first cast as cast iron and later made malleable without remelting.

Remarks. Commercial malleable-iron castings are first cast as brittle white cast iron, and are then made more or less malleable either by converting most of their carbon from the state of cementite into that of temper graphite, or by removing most of it by oxidation, or by both means jointly. In both cases the malleabilizing is done by close annealing, usually in contact with an oxidizing agent. Thus, there are two classes of commercial malleable-iron castings, of which one owes its malleability to a large removal of carbon, and the other to a large removal of carbon from the outer part of the casting, and to the precipitation of much of the remaining carbon in the free or graphitic state. It is not in accordance with good industrial usage to apply the name "steel" to any product of these malleabilizing processes, or to any product made by remelting cast iron in a cupola or like furnace, and such application is generally held to be fraudulent.

Malleable-iron castings differ from steel in being cast into a mass which is not initially malleable. Malleable-iron castings are not of cast iron, but are made from cast iron, and are as distinct from cast iron as wrought iron and steel are.

\* See *Proc. I. A. T. M.*, Sixth Congress, 1912, p. 145 of Sec. A.

**645. Wrought Iron.**—Definition. Malleable iron which is aggregated from pasty particles of metallic iron without subsequent fusion, and contains so little carbon that it does not harden usefully when cooled rapidly.

Remarks. Commercial wrought iron, though occasionally made directly from the ore, is usually made from cast iron by such removal of its carbon and silicon as to convert it, at the high temperature used, into pasty particles, and by squeezing these together in a bath of cinder or slag into a coherent mass which retains permanently an important quantity of that slag. The fact that some of the particles from which such a malleable product is thus aggregated are initially of steel, does not in itself remove such a product from the class "merchantable wrought iron." But if all of such particles were initially of steel and none of them initially of wrought iron, the product is not now recognized as merchantable wrought iron, but is called industrially "scrap steel."

In Great Britain "malleable iron" is often used as synonymous with "wrought iron," but sometimes at the risk of leaving the lay reader in doubt whether it refers to wrought iron or to malleable-iron castings. Hence we advise against this use of "malleable iron."

**646. Steel.**—Official Definitions. See "Carbon Steel" and "Alloy Steel" below.)

**1. Fluid Origin.**—Definition. Iron which is cast from the molten state into a mass which is usefully malleable initially at least in some one range of temperature. (See "Steel II," below.)

Remarks. Such metal is steel whether it can be hardened or not, whether it contains much or little or even no carbon, and for that matter even if it is chemically pure iron. It is sometimes called "ingot metal." With the exception of "blister steel" and its derivatives all the steels which have any present industrial importance including the "alloy steels" (see below) fall under this definition.

These steels have been divided into:

(a) *Ingot Iron.*—Steel with too little carbon to harden usefully on rapid cooling. This name has never been widely used, and for a long time fell into complete disuse. It has lately been revived as a trade name. It should be avoided in scientific and technical writings because, with the iron-carbon alloys divided into the four great species, (1) wrought iron, (2) steel, (3) cast iron, (4) malleable iron castings, it is confusing to call a variety of "steel" "ingot iron."

(b) *Ingot Steel.*—Steel with enough carbon (say 0.30 per cent or more) to harden usefully on rapid cooling.

Steel made by melting in a crucible is called "crucible steel"; that made in an electric furnace is called "electric steel."

**Steel II: Plastic Origin.**—Definition. Iron which is aggregated from pasty particles without subsequent fusion; is malleable at least in some one range of temperature; and contains enough carbon (say 0.30 per cent or more) to harden usefully on rapid cooling from above its critical range.

Remarks. Blister steel and its derivatives and a few other high carbon steels which are important to-day, are the only present steels covered by this definition.

The British Members are unable to accept the official definitions and accept the following alternatives:

*Steel:* Iron either pure or associated with other elements which contains sufficient carbon to cause it to harden usefully when rapidly cooled from high temperatures and is so malleable initially, that it can be forged or rolled; also iron which has been cast from a molten state, which does not contain sufficient carbon to cause it to harden usefully when rapidly cooled, and is so initially malleable that it can be forged or rolled.

**647. Blister Bar, Cement Bar, Converted Bar.**—Official definitions. Steel of plastic origin made by cementing wrought iron with carbonaceous matter. Also,

commercially, such steel when heated and worked into merchant sizes. Most writers have used "blister steel" in the former and broader sense. In Sheffield it is used solely in the latter and narrower sense.

**Remarks.** The blister steel of commerce is made by cementing very pure wrought iron with charcoal. The term Cemented Bar, may be applied in a general sense to any wrought-iron bar which has been subjected to a process of cementation.

**648. Blister Steel.**—Bars rolled or forged from blister bar. Many writers have used "blister steel" in the sense of "blister bar."

The British Members are unable to accept the official definitions and propose the following alternatives:

*Blister Bar, Cement Bar, Converted Bar:* Definition. High class Swedish or other wrought-iron bar of equal purity which has been subjected to a process of cementation in contact with charcoal, while introducing carbon into the iron also develops blisters on the surface of the cemented bar.

*Blister Steel:* Definition. A term sometimes used to describe cemented converted or blister bar which has been heated and worked into merchant sizes.

**649. Plated Bar.**—Definition. Bars of blister steel which have been rolled or hammered while hot.

**Remarks.** This treatment, which is usually applied to such bars broken to convenient lengths, flattens down their blisters and toughens the metal somewhat.

**650. Single-shear Steel.**—Definition. Shear steel made by welding a pile of plated bars into a faggot. Also the bars and other merchant shapes made by rolling of hammering such a faggot.

**651. Double-shear Steel.**—Definition. Shear steel made by piling, hammering, and thus welding bars of single-shear into a bloom. Also the bars and other merchant shapes made by rolling or hammering such a bloom.

**652. Carbon Steel.**—Definition. Steel which owes its distinctive properties chiefly to the carbon as distinguished from the other elements which it contains.

**Remarks.** Though among the alloy steels some are but moderately malleable, among the carbon steels industrial usage confirms the name "steel" to products malleable enough to be rolled or forged into merchant shapes.

**653. Alloy Steel.**—Definition. Steel which owes its distinctive properties chiefly to some element or elements other than carbon, or jointly to such other elements and carbon.

**Remarks.** Some of the alloy steels necessarily contain an important percentage of carbon, even as much as 1.25 per cent. There is no agreement as to where the line between the alloy steels and the carbon steels shall be drawn.

**654. Ferro-alloys.**—Definition. Iron so rich in some element or elements other than carbon that it is used primarily as a vehicle for introducing that element in the manufacture of iron or steel.

**Remarks.** The ferro-alloys are not usually malleable, and they usually contain more of the alloying element than is desirable in an alloy steel. With variations in industrial conditions the line between alloy steels and ferro-alloys must needs shift. Indeed, a substance might simultaneously be an alloy in the machine shop and a ferro-alloy in the steel mill.

**655. Semi-steel.**—A vague trade name for various products near the border line between steel and cast iron. Among these are low carbon cast iron made in the air furnace, or in the cupola furnace by the addition of steel scrap to the charge. Also a trade name for malleable cast iron.

While the report failed of adoption, because of considerable opposition, and does not represent, therefore, the official view of the Associa-

tion, it conforms very closely with the consensus of opinion in this country. The objections were raised largely by Germans, and by manufacturing interests in this country, who have been making an open-hearth product of exceptional purity, the trade value of which has been much enhanced by its designation as iron.

## COMPOSITION AND CONSTITUTION

**656. Composition of Iron and Steel.**—Chemical composition of typical forms of iron and steel products finding utility in connection with engineering construction, are given in Table 1.

TABLE 1.—TYPICAL COMPOSITIONS OF CERTAIN IMPORTANT FERROUS METALS

METAL.	COMPOSITION IN PER CENT (IRON OMITTED).								
	C	Si	Mn	S	P	Ni	Cr	Va	Ti
Ingot iron (pure).....	.01-.02	< .01	.01-.02	.01-.02	< .01				
Boiler plate.....	10-.20	.05-.15	.30-.50	< .05	< .05				
Structural steel.....	15-.25	.05-.25	.30-.50	.04-.06	.04-.10				
Structural steel (electric)	15-.25	.05-.25	.50-.80	< .02	< .02				
Axles and shafts.....	.30-.40	.05-.25	.40-.80	< .05	< .05				
Vehicle springs.....	.90-1.0	.05-.25	.25-.50	< .05	< .05				
Rails.....	.40-.70	< .20	.60-1.0	< .06	< .10				
Carbon tool steels.....	.50-1.5	.10-.30	.10-.50	< .03	< .03				
Steel castings.....	15-.40	.20-.35	.50-1.0	< .05	< .06				
Cast-iron castings.....	2.5-4.0	.70-2.5	.40-1.0	< .12	20-1.2				
Nickel steel (structural)	.30-.45	.05-.25	.60-.70	< .05	< .05	3.5-4.0			
Manganese steel.....	1.0-1.3	.30-.80	11-14		< .06				
Transformer (silicon) steel	.05-.10	.40-4.3	10-.15	< .06	< .05				
Vanadium steel.....	.25-.50	.10-.25	.80-.90	< .05	< .05			.15-.25	
Titanium rail steel.....	.40-.80	.05-.10	.80-1.0	< .05	< .05				.10-.20
Nickel-chrome steel.....	.20-.40	< .10	.30-.60	< .05	< .05	1.2-4.0	.60-2.0		
Chrome-vanadium steel.....	.20-.60	.10-.25	.50-.80	< .04	< .04		.80-1.1	.15-.25	
High-speed steels *.....	.60-.80	.15-.40	0-2.0	< .03	< .03		3.0-5.0	.50-2.0	W 13-19

\* Some high-speed steels also contain 3 to 5 per cent of cobalt.

While commercial forms of iron and steel have in general several elements associated with the iron, and each of these has some effect on the physical properties, the best method of treatment in the present state of our knowledge is to consider the material as essentially an alloy of iron and carbon; the inherent properties of this binary series are modified by the other associated elements.

**657. Determinations of Constitution of Iron and Steel.**—The pioneer attempts at grouping the iron-carbon series into a constitution diagram were made by Sauveur, in 1896, and Roberts-Austen, in 1897. In 1900, Roozeboom constructed a diagram based upon experimental data and interpreted in accordance with the "Phase Rule." The data were accurately redetermined and somewhat elaborated by Carpenter and Keeling, in 1904. While some modifications of the original have been necessary to bring the theoretical deductions in line with well-established facts of practice, and while there has been a very great amount of

controversy over certain details of interpretation with new or modified diagrams to harmonize with these various conflicting views, nevertheless, the "Roozeboom Diagram" has remained as the rational basis of our understanding of iron and steel.

**658. Critical Temperatures.**—Iron is a relatively complex element. Between its freezing-point of  $1505^{\circ}$  C. and normal temperatures, it exhibits transition points at  $900$  and  $760^{\circ}$  C., which are indicative of allotropic states of the iron, the so-called "alpha" (below  $760^{\circ}$ ) "beta" ( $760$  to  $900^{\circ}$ ) and "gamma" (above  $900^{\circ}$ ) modifications. The existence of beta iron is denied by some authorities; the chief manifestation in pure iron is the slight break in the cooling curve at the critical temperature of  $760^{\circ}$  C., and the loss or gain of magnetism by the iron when heated above, or cooled below that temperature. No change in the crystallographic form of the iron has been observed. The transition from beta to gamma iron at  $900^{\circ}$  C. is accompanied by a change in the crystallographic form of the iron, which, in turn, affects the solubility relations of the iron and carbon, and, therefore, has a most vital bearing upon the structure and physical properties. The critical temperatures in the vicinity of  $760^{\circ}$  and  $900^{\circ}$  are called the  $A_2$  and  $A_3$  points, respectively. Due to lag in reaction, they are somewhat higher on rising temperature than if recorded on cooling curves; the designations  $A_{c2}$  and  $A_{c3}$  are used for the former, and  $A_{r2}$  and  $A_{r3}$  for the latter. In the iron-carbon alloys there is another important critical temperature  $A_1$  ( $A_{r1}$  on cooling  $A_{c1}$  on heating) in the vicinity of  $700^{\circ}$  C. (see Art. 660).

**659. Cementite, Ferrite and Graphite.**—Iron forms with carbon a compound having formula  $Fe_3C$ , technically classified as cementite, and often called iron carbide or combined carbon. It contains 6.67 per cent of carbon and 93.33 per cent of iron, and is extremely hard and brittle; in fact the hardest constituent of steels, about 6 to  $6\frac{1}{2}$  compared with 4 to 5 for iron in Mohs' scale.

In metallographic nomenclature the carbonless iron is called ferrite.

Carbon in the form of small flakes or thin plates, which are mechanically mixed with the iron, is called graphite.

**660. Alloying Relations between Iron and Cementite at High Temperatures.**—For purposes of discussion it is best to consider the alloying relations between iron and cementite ( $Fe_3C$ ) as the constituents of the binary series. For each 1 per cent of carbon there is 15 per cent of  $Fe_3C$  formed, and the limit of the series will be at 6.67 per cent carbon, or 100 per cent of the  $Fe_3C$ . The solidification diagram is shown in Fig. 1. The melting-point of pure iron is taken as  $1505^{\circ}$  C.; that of cementite is not known, since it has not been isolated in mass, and it undergoes decomposition before fusion. It is

not material to the discussion, however, since the useful range of composition does not exceed 5 per cent of carbon content.

Consider first only that part of the diagram above  $1100^{\circ}\text{C}$ . It is typical of complete solubility of iron and carbon (or iron and  $\text{Fe}_3\text{C}$ ) in the liquid condition, with partial solubility in the solid, at least at the iron end of the series. Addition of either constituent to the other lowers the freezing-point of the alloy to a eutectic temperature and composition  $E$  of  $1130^{\circ}\text{C}$ .

and 4.3 per cent carbon (64.5 per cent  $\text{Fe}_3\text{C}$ ), respectively. The limit of solid solubility is at  $S$ , that is, 2 per cent of carbon, or 30 per cent of cementite, at  $1130^{\circ}\text{C}$ .; this solubility decreases, however, with further lowering of temperature as is indicated by the slope of the line  $PS$ .

There are six characteristic areas for the part of the diagram under discussion, as follows:

I. The alloy is entirely molten, and is a liquid solution of iron and carbon, or of iron and cementite.

II. A region of selective freezing, with primary crystallization of a solid solution of iron and  $\text{Fe}_3\text{C}$ .

III. A region of selective freezing, in which there is primary separation of cementite as the excess substance.

IV. Completely solid homogeneous alloys, consisting of solid solutions of gamma iron and cementite with a saturation limit of 30 per cent of the latter at  $1130^{\circ}\text{C}$ . This solid solution is technically called austenite.

V. Completely solid alloys, exhibiting heterogeneity of structure, with primary crystallization of saturated austenite, together with an eutectic consisting of a composite of saturated austenite plus cementite.

VI. Completely solid alloys, exhibiting heterogeneity of structure, with primary crystallization of cementite, together with an eutectic consisting of a composite of saturated austenite plus cementite.

An alloy of 4.3 per cent carbon (64.5 per cent  $\text{Fe}_3\text{C}$ ) should consist solely of the eutectic of austenite and cementite.

**661. Allotropic Modifications in Cooling.**—The complexity of the iron-carbon series results from the change in the solubility relations of

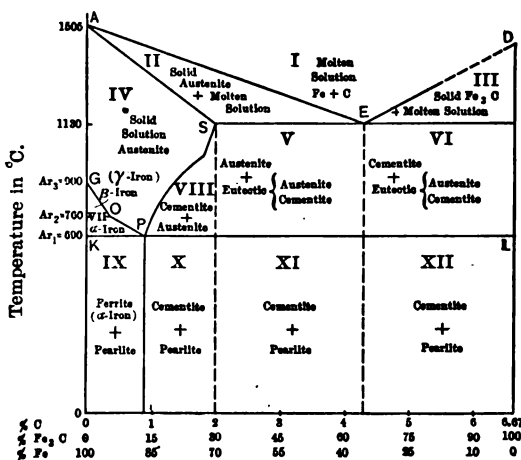


FIG. 1.—Roozeboom Diagram (slightly modified).

the iron and cementite in the completely solid alloys, due to the allotropic modification at  $900^{\circ}\text{C}$ . Gamma iron will hold  $\text{Fe}_3\text{C}$  in solid solution up to a limiting content of 30 per cent at  $1130^{\circ}\text{C}$ .; whereas the cementite is insoluble in the beta and alpha modifications below  $1100^{\circ}\text{C}$ .; therefore the Roozeboom diagram becomes complicated by reason of the breaking down of the austenite in a manner entirely analogous to that of a molten alloy where solid solubility is nil. When the metal is cooling the eutectic point  $P$  (called eutectoid to distinguish it from the other one at  $E$ ) is at a temperature of  $690^{\circ}\text{C}$ . ( $A_{r1}$ ) and a carbon content of 0.9 per cent, or 13.5 per cent of  $\text{Fe}_3\text{C}$ . On heating the corresponding critical temperature  $A_{c1}$  is somewhat higher due to lag in the reaction. This eutectoid is a composite mixture of 13.5 per cent of cementite and 86.5 per cent of ferrite, and is technically designated as pearlite.

Austenite of composition below 0.9 per cent carbon will undergo decomposition in cooling through region VII; this will begin when the temperature reaches the point on  $GOP$  corresponding to the composition of the alloy, and will be completed when it has fallen to  $690^{\circ}\text{C}$ . During the period of selective crystallization, there will be primary separation of ferrite, with a progressive shift of austenite composition with such selection and fall of temperature, until the eutectoid  $P$  is reached. The austenite now consists of 86.5 per cent of ferrite and 13.5 per cent of cementite in solid solution. Attempt to lower the temperature below  $690^{\circ}\text{C}$ . will result in the splitting up of this solution to a granular or lamellar eutectic of ferrite and cementite, each physically distinct. The resultant alloy at normal temperatures will consist of primary ferrite plus the pearlite eutectic.

The relative proportions will depend upon the carbon content of the alloy; the quantity of pearlite will increase from 0 to 100 per cent as the carbon content increases from 0 to 0.9 per cent, corresponding to a variation of cementite from 0 to 13.5 per cent. An alloy of 0.45 per cent carbon will exhibit a structure of one-half excess ferrite plus one-half pearlite.

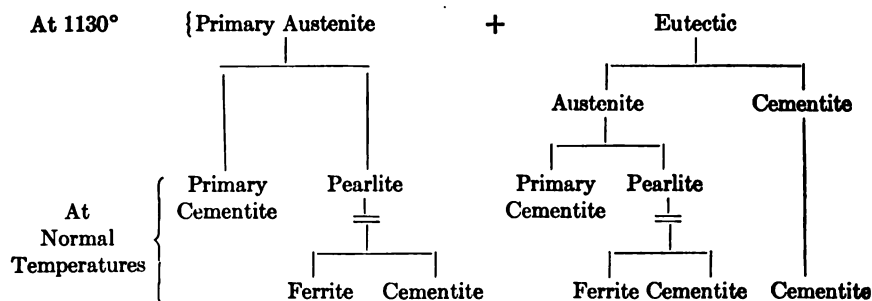
Should the austenite have a composition of 0.9 to 2.0 per cent of carbon, selective decomposition will take place in region VIII, beginning at a temperature along  $PS$ , and ending at  $690^{\circ}\text{C}$ . There will be primary separation of the excess cementite with final formation of the characteristic pearlite. The final alloy will consist of cementite plus pearlite; the proportion of the latter decreasing with increasing of carbon above 0.9 per cent. With a carbon content of 1.5 per cent, the structure will consist of 10 per cent of excess cementite, and 90 per cent pearlite.

If the alloy under consideration has a carbon content of 0.9 per

cent, no change in the austenite will occur until the eutectoid temperature of  $690^{\circ}$  C. is reached, at which time there will be a simultaneous separation of cementite and ferrite from the solid solution. The resultant alloy will consist entirely of pearlite. This pearlite will be of granular or lamellar texture, depending in a large measure upon the conditions of cooling. The first stages of decomposition of the austenite will result in finely granular particles of free ferrite and free cementite, because of lack of solubility below the critical temperature of  $690^{\circ}$  C. Should the rate of cooling below  $690^{\circ}$  be very slow, or especially if the temperature is held slightly below the eutectoid for considerable time, coalescence of the ferrite granules with their like, and of cementite granules with their like, will take place with a resultant lamellar structure characteristic of well annealed steels.

All iron-carbon alloys of above 2 per cent of carbon content have a heterogeneity after completion of solidification at 1130°, as noted heretofore. At this time, for compositions between 2 and 4.3 per cent of carbon, the structure is saturated austenite (2 per cent carbon) of primary crystallization, plus a eutectic of saturated austenite and cementite. The relative quantity of eutectic increases in proportion as the carbon content of the alloy under consideration approaches 4.3 per cent. On lowering the temperature below 1130° C. the cementite alone is stable; the primary austenite as well as that of the eutectic, undergoes selective crystallization with decrease of temperature. Cementite separates out of solid solution, with progressive shift of temperature and composition of remaining austenite, until the eutectoid at 690° C. and 0.9 per cent carbon is reached. Here the eutectoid austenite undergoes transition to pearlite.

Alloys between 2 and 4.3 per cent of carbon undergo transition as follows:



Thus at ordinary temperatures, two constituents only, the cementite and ferrite, are present; yet there is complexity of structure because the cementite may have five degrees of aggregation due to the selective





regions V and VI breaks up less readily, and that in the pearlite of regions XI and XII is decomposed with difficulty. With the rate of cooling and carbon content constant, the proportion of graphite will vary with the percentage of silicon. If the silicon content and rate of cooling are constant graphitization will increase with the percentage of carbon. Again, if the composition remains constant more carbon will be freed by slow cooling than by rapid cooling.

Thus, depending on the proportion of silicon and the rate of cooling, a high-carbon alloy may have any one of the following constitutions:

1. Cementite plus pearlite;
2. Cementite plus pearlite plus graphite;
3. Pearlite plus graphite plus ferrite;
4. Graphite plus ferrite, which is the only stable constitution but is also very rare.

**663. Structures in Iron Carbon Alloys.**—Fig. 2 gives in diagrammatic form the structural relations of the iron-carbon series. For 0.3 per cent of carbon, the relative proportions of the constituents will be two-thirds excess ferrite and one-third pearlite; for 0.6 per cent carbon, one-third ferrite and two-thirds pearlite; for 0.9 per cent carbon, all pearlite. Above 0.9 per cent carbon, cementite becomes the excess constituent of primary separation. An alloy of 2.0 per cent carbon will consist of 20 per cent excess cementite and 80 per cent pearlite; while with 3.8 per cent of carbon, the structure will show one-half cementite and one-half pearlite. The pearlite is in all cases of constant composition, 86.5 per cent of ferrite and 13.5 per cent of cementite; and the total ferrite or cementite in the alloy will be that constituting the excess substance—either ferrite or cementite alone—plus the amount in the pearlite.

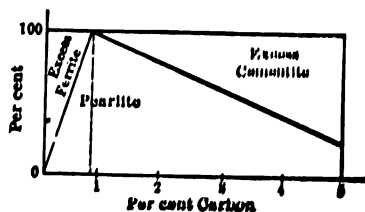
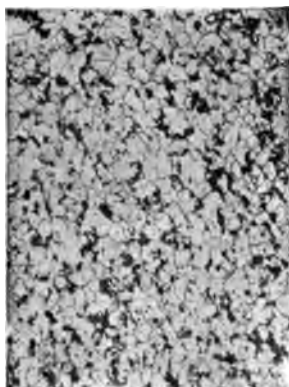


FIG. 2.

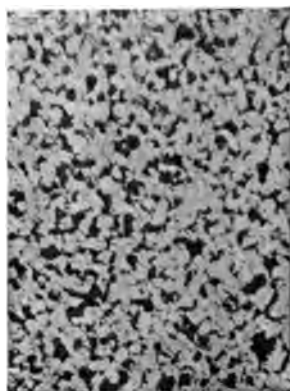
Typical microstructures of the iron-carbon series are given in Fig. 3 and 4. The etching has been with picric acid solution; this reagent does not attack cementite, and ferrite in mass but very slowly. Therefore, they retain the mirror-like reflecting surface of the polished specimen and appear white in the photographs. However, it does attack the ferrite of the pearlite, probably because of its fine state of division and the electrolytic action set up with the closely associated cementite. Consequently since the ferrite constitutes 86.5 per cent of the pearlite, and the latter is usually in such a fine state of aggregation that under lower magnification (50 diameters or less) the separate constituents are



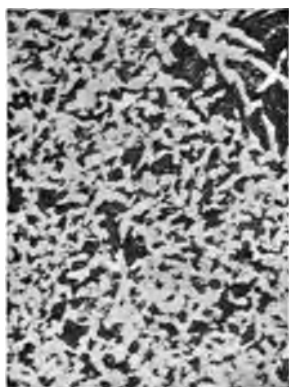
(a) Ingot Iron, C. = 0.00%.



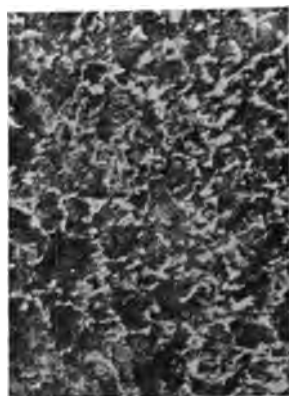
(b) Steel, C. = 0.10%



(c) Steel, C. = 0.27%



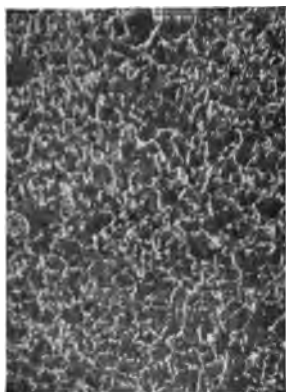
(d) Steel, C. = 0.45%.



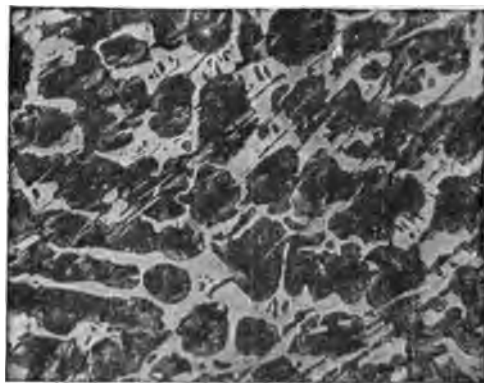
(e) Steel, C. = 0.74%.



(f) Steel, C. = 0.85%.



(g) Steel, C. = 1.5%.

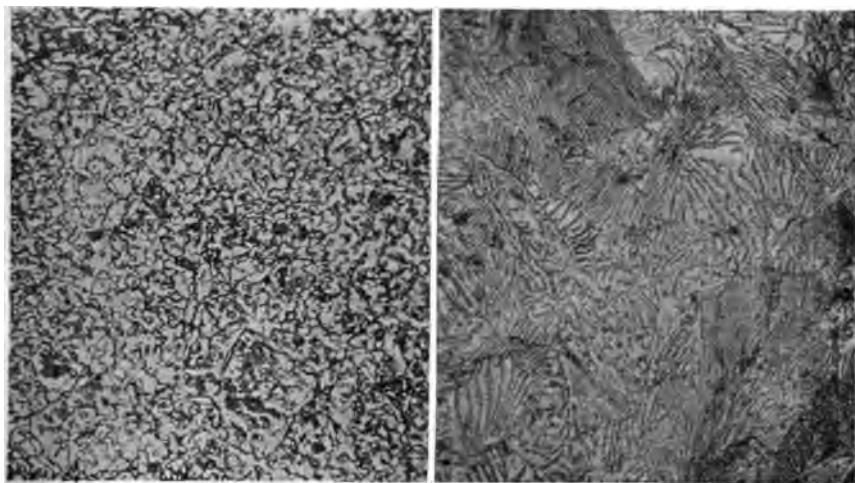


(h) White Cast Iron, C. = 3.5%.

FIG. 3.—Structures of Iron-Carbon Alloys Slowly Cooled.

Black constituent in (b), (c), (d), (e), (g) and (h) is pearlite; White is ferrite in (b), (c), (d) and (e), cementite in (g) and (h). In (f) is shown lamellar pearlite, here the black is ferrite and the white cementite. Fig. 3f by Tinsley; others by Aston. Magnification in f = 1000 diameters; in others, 70.

**not** revealed, the pearlite is apparently entirely etched, and then **appears** black in the photograph, due to the dispersion of the light by the roughened surface.



(a) Magnification = 410d.

(b) Magnification = 1000d.

**FIG. 4.**—Globular Pearlite (a) and Lamellar Pearlite (b). (Courtesy of Sauveur and Boylston).

Higher magnifications (500 to 1500 diameters) reveal the composite nature of the pearlite, see globular pearlite in Fig. 4a and lamellar type in Fig. 4b. The black constituent is the ferrite and the white, cementite.

## CHAPTER XXII

### PROPERTIES OF WROUGHT IRON

**664. Structure.**—Under the microscope wrought iron is seen to consist of grains of ferrite more or less surrounded by filaments of slag. In the cross-sectional view of Fig. 1a the slag inclusions appear as dark rounded patches of variable size, whereas in the longitudinal section, Fig. 1b, their thread-like nature is evident. The amount of slag in

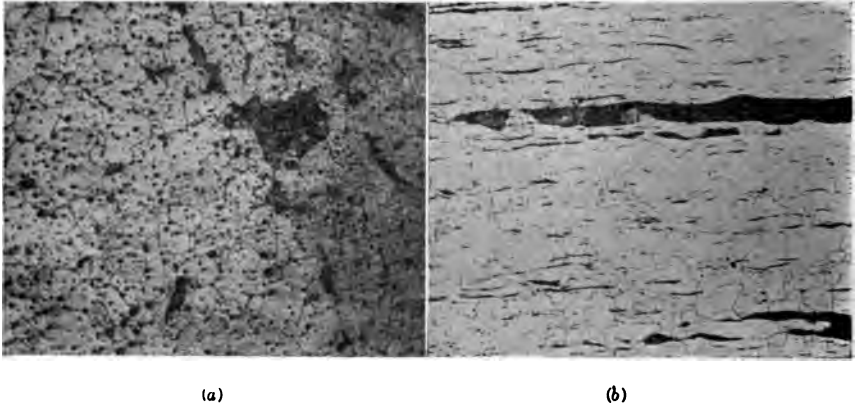


FIG. 1.—Photomicrographs of Wrought Iron. (a) Section Transverse to Direction of Rolling; (b) Section Parallel to Rolling. (Courtesy of Sauveur and Boylston) Magnification, 50 Diameters.

the iron depends upon the character of materials charged into the furnace, upon the work received by the puddle ball and upon the amount of hot work done in shaping the iron. The size of the ferrite grains is dependent upon the time the metal is held at temperatures above the critical range and upon the temperature from which it cools. High temperatures and long soaking periods favor large grains. Rapid cooling and continued hot working of the iron while it is above the recalcrescence point effect small grain size. To secure maximum fineness of grain the hot work should not cease until the temperature has fallen to about 700° C. The ductility and toughness of the iron are rendered high by removing the slag and by making the grain size as fine as possible without causing permanent distortion of the grains.

When, however, a wrought-iron bar of uniform section is ruptured in the ordinary tensile test it generally exhibits an irregular fibrous fracture (Fig. 6, Ch. III). Occasionally, if the iron has been made from steel scrap or if it has not been properly worked during the "boil," the tension fracture will be partially or wholly crystalline. On the other hand if wrought iron fails under shock or under repeated loading it generally has a crystalline fracture. This circumstance has led some to the erroneous conclusion that the iron crystallizes in service due to vibrations or shock. As a matter of fact the structures of all ferrous metals are crystalline. Other indications sensed by the naked eye are due to the drawing out of the grains by the method of testing to such extent that they are no longer visible as such but appear like fibers,—or, in soft steel, like fine silk. Thus, good wrought iron normally presents a distinct fibrous fracture in tension due to the presence of the brittle slag filaments which imperfectly surround groups of iron grains. The latter are drawn out in the test, but the slag filaments, being more brittle than the ferrite, cause minute planes of discontinuity which by contrast make the bundles of iron grains appear like fibers. In soft steel where no slag is present there is no grouping of iron grains; the structure is more homogeneous, and the fracture, being much finer and more uniform in texture, appears silky.

The occasional crystalline fracture of wrought iron under static tension is generally due to the presence of particles of steel or cast iron in the path of the break. These particles on rupturing exhibit their normal structure.

The crystalline fracture under shock is due to the suddenness of the application of the force, time not being given for more than partial distribution of the induced stresses to adjoining sections or for drawing out of the grains at the break.

When a bar of wrought iron containing considerable steel scrap is nicked with a chisel or grooved with a tool and broken transversely, it is likely to show a crystalline or partially crystalline fracture, although in tension the fracture may be fibrous. The crystalline fracture is due to the stress concentration at the root of the nick and the reinforcement afforded by the sides of the nick. This condition causes a large stress to be exerted on a comparatively few grains without allowing elongation, and a break squarely across the grains ensues. The scrap steel portions of the bar containing little slag will, when fractured, appear crystalline.

The crystalline failure under repeated stress is readily accounted for by the formation of slip bands as explained in Art. 823.

**665. Defects.**—The evil influences of phosphorus and sulphur appear to be less pronounced in wrought iron than in steel. This may be due

to the fact that there is less opportunity for segregation in the puddling process than in processes where the metal is cast from a molten state. Furthermore, much of the impurities in wrought iron are affiliated with the slag rather than the iron. However, a very high sulphur content (0.3 to 0.5 per cent) is likely to cause the iron to crumble, or exhibit what is known as red shortness, in forging or welding. Sections rolled from *red short* iron are likely to have rough edges. A phosphorus content of 0.4 to 0.5 per cent, if the phosphide surrounds the ferrite grains, causes the iron to be brittle at cold temperatures (*cold short*) and renders it hard and coarsely crystalline. Such iron cannot be refined by heat treatment.

In sheets or plates spongy spots, often called *spilly places*, are occasionally found. These defects are due to burning of portions of the iron in puddling. *Blisters* are also found on plates and sheets made of inferior iron. They appear to be due to the oxidation of carbon in the iron by the oxide of iron in the slag.

666. The tensile strength of wrought iron along the grain is practically the same as the tenacity of ferrite, and varies from 45,000 to 55,000 lb. per square inch. It is greater in small rods and thin plates than in large bars and thick sections, the material remaining the same. This is shown in Fig. 2, where the same material has been rolled into bars from  $\frac{3}{8}$  in. to 2 in. in diameter, the tensile strength varying from 52,000 in the smaller to 47,500 lb. per square inch in the larger sizes. The increase in strength is probably due to the reduction in the size of the ferrite grains which is favored by a large amount of hot work and is attended by an increase in density and cohesion between grains.

The *Elastic Limit* is more dependent on the thinness of the final section than on the tensile strength, as is well brought out in Fig. 2. Here the apparent elastic limit varies from 40,000 lb. per square inch in the  $\frac{3}{8}$ -in. rods to 23,000 lb. per square inch in the 2-in. rods, and is almost identical with the "yield-point." This increase in the elastic limit with increased reduction in the rolls always occurs with wrought iron and steel, but it is much more pronounced with wrought iron. The true elastic limit of wrought iron is nearly always much lower than the apparent elastic limit. In Fig. 2 it is found from 5000 to 7000 lb. lower in every case. In mild steel these two limits are almost identical.

The *Percentage of Elongation* in 8 in. varies from 5 per cent to 25 per cent when tested in the direction of the fibers, depending on the quality of the material, the reduction of area averaging about 50 per cent more than the elongation. The elongations recorded in Fig. 2 were all taken on a length of 20 in., which somewhat reduces the per-

centage, especially for smaller sections. Iron with high slag content will exhibit less elongation and far less reduction in area than the best grades of Swedish iron which run low in slag.

*The pulling speed* appears to have little effect on the strength or ductility of wrought iron when the test-pieces are of standard dimensions. In *Tests of Metals*, 1887, p. 924, there are data to show that high speeds, say 5 in. per minute, will raise the strength of grooved specimens 4 per cent and the strength of specimens with short gage lengths, about 2 per cent above the values for very slow speeds. The ductility of such specimens is decreased by rapid application of stress.

667. The tensile strength across the grain is always much less than along the grain for wrought iron, whereas with steel there is no appreciable difference. From Bauschinger's *Communications*, Vol. 2, the following data were gotten.

The ratio of lateral to longitudinal tensile strength from eight tests in each direction on iron of an exploded boiler was 0.74. From the

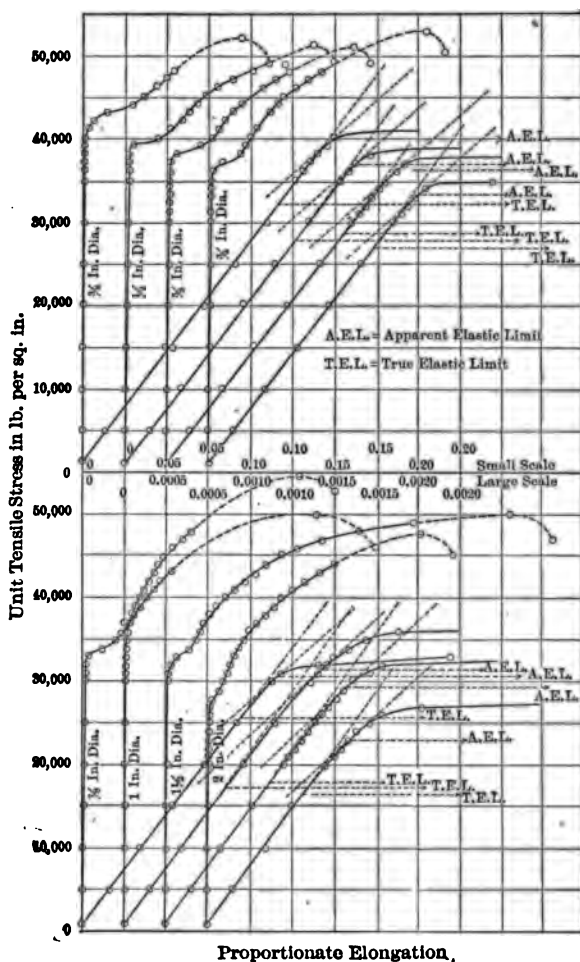


FIG. 2.—Stress-diagrams (in Tension) of Wrought-iron Bars of Varying Diameters, all Rolled from the Same Material. All elongations measured on a length of 20 in. The "Apparent Elastic Limit" falls from 2% to 10% lower than the indicated "Elastic Limit" in the original report; it varies from 23,000 in the 2-in. to 40,000 lb. in the  $\frac{3}{8}$ -in. specimens; and it marks a point where the permanent set is less than 0.0001 of the length of the specimen. Each diagram is the average of from 3 to 6 tests. (*Tests of Metals*, 1888.)



same number of like tests on iron of a second boiler the mean ratio 0.71 was computed. Results of tests on six new plates from as many different sources yielded ratios of 0.76, 0.62, 0.92, 0.90, 0.76 and 0.83.

It is asserted that by cross piling of the "puddled lumps" (small slabs corresponding to muck bar) the best Lowmoor iron plate of Yorkshire, England, is given an ultimate strength parallel to the rolling of 51,500 lb. per square inch and elongation of 16 per cent. Across the grain the strength is 45,000 lb. per square inch and the elongation

12 per cent. This material is about four times as costly as the best of steel plates consequently its use for purposes where forging and welding are not required is declining.

In conclusion we may say that the tensile strength of wrought iron transverse to the direction of the rolling ranges from 60 to 85 per cent of its strength parallel to this direction.

**668. The compressive strength of wrought iron**, like that of other ductile metals, must be regarded as the yield point. At this stress the metal buckles out of shape; and, if the specimen has appreciable length, failure follows. It is likely that thin sections which receive more work during rolling will be stronger than thick sections. Assuming the yield points of wrought iron in tension and compression to be

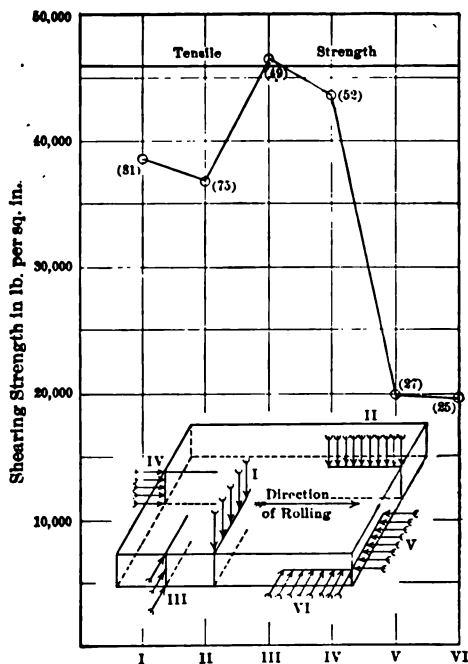


FIG. 3.—The Shearing Strength of Wrought Iron on the Six Principal Planes, as Compared with its Tensile Strength. Figures indicate the number of results averaged. (*Bauschinger's Communications*, Vol. II).

the same, we see from Fig. 2 that the ultimate strength in compression of short sections  $\frac{1}{4}$  to  $\frac{5}{8}$  in. in thickness will vary from 40,000 to 30,000 lb. per square inch. Since the elastic limit of thin steel sections is not increased so much by reduction in rolling, it becomes apparent why many of the wrought-iron columns built of very thin material have, when tested, exhibited ultimate strengths nearly as great as columns of mild steel.

**669. The Shearing Strength of Wrought Iron.**—A very elaborate investigation of the properties of wrought iron was reported by Bau-

schinger in his *Communications*, Vol. 2. He made several hundred shearing tests on wrought-iron plate from seven different sources and found the shearing strengths in two directions on each of three principal planes as shown in Fig. 3. Since there was a general agreement in the relative strength on these planes, only the averages for a portion of the tests are given in the diagram. In general, we may say that the shearing strength across the thickness of the plate, either with or across the grain, is about 80 per cent of the tensile strength, while if the external forces are parallel to the plane of the plate, and are applied on the planes perpendicular to the plane of the plate, the shearing strength is about the same as the tensile strength. The shearing resistance on a plane parallel to the plane of the plate is less than 45 per cent of the tensile strength.

670. The modulus of elasticity of wrought iron lies between 25,000,000 and 29,000,000 lb. per square inch for either tension or compression. Iron of good quality usually has a modulus of approximately 27,000,000 lb. per square inch. Values reported by C. A. Marshall in *Trans. Am. Soc. Civ. Engr.*, Vol. 17, p. 62, are given in Table 1.

TABLE 1.—VALUES OF THE MODULUS OF ELASTICITY OF WROUGHT IRON  
(IN 1000 POUNDS PER SQUARE INCH)

Size of Bar.	TENSION.		COMPRESSION.		Size of Bar.	TENSION.		COMPRESSION.	
	$E_1$ First Loading.	$E_2$ Second Loading.	$E_1$ First Loading.	$E_2$ Second Loading.		$E_1$ First Loading.	$E_2$ Second Loading.	$E_1$ First Loading.	$E_2$ Second Loading.
$\frac{3}{4}$ rd.	26,800	27,500	25,840	26,160	1 sq.	28,290	28,290	27,100	27,990
$\frac{1}{2}$ rd.	26,980	27,410	25,920	26,240	1 rd.	27,590	28,570	27,250	28,570
1 rd.	.....	26,700	25,670	26,440	1 rd.	28,290	28,480	27,430	28,570
1 rd.	27,540	27,540	26,020	26,350	1 rd.	26,580	28,480	26,500	28,180
$\frac{3}{4}$ sq.	28,990	.....	27,420	.....	$\frac{1}{2}$ rd.	30,190	30,190	29,520	29,910
$\frac{1}{2}$ sq.	27,790	29,180	25,650	27,790	mean	27,894	28,203	26,734	27,590
1 sq.	27,800	27,900	26,490	27,300					

They indicate that the modulus is slightly greater after the specimen has been strained than on first loading. The modulus of elasticity in torsion is approximately 11,000,000 lb. per square inch.

671. **Effects of Overstrain.**—As in steel and other ductile metals so in wrought iron, when strained beyond its elastic limit and allowed to rest the elastic limit and ultimate strength are raised but the elasticity under stress of opposite kind is impaired. Fig. 4 shows the effects of overstraining three 3-in.  $\times$  1-in. wrought-iron bars. The original bars

were 100 in. long between gage points. They broke at an average ultimate strength of 53,200 lb. per square inch with an elongation of 16 per cent in 100 in. The second tests were made on the broken halves using a 50-in. gage length. It will be noted that after several months rest the elastic limit, computed on the original area, is higher than the initial ultimate strength of the second test. The elongation has, however, been markedly reduced. Annealing of the overstressed pieces reduced the elastic limit and ultimate strength below the original values. Stretching the iron slightly beyond the elastic limit

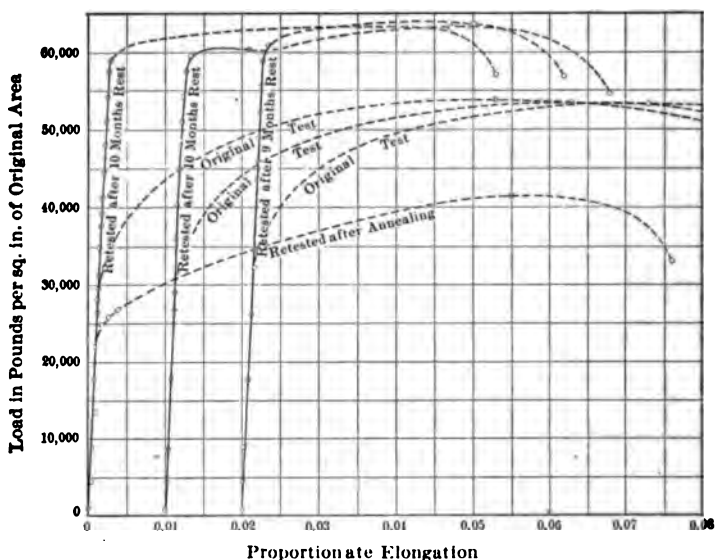


FIG. 4.—Wrought Iron Bars, 3 by 1 by 100 In., Retested. First test gave  $S_e = 30,000$  lb. per in.<sup>2</sup>;  $S_t = 53,700$  lb. per in.<sup>2</sup>; per cent elong. in 100 in. = 16. retests were made on the ends of the broken bars. Elongations were measured in 50 in. and unit stresses were computed in terms of original areas. (*Tests of Metals*, 1882.)

would cause less marked changes in strength and ductility. Fig. 5 shows that the magnitude of the effect of overstrain is dependent to some extent upon the subsequent period of rest.

Fig. 6 illustrates the loss of elasticity suffered by wrought iron due to reversal of stress immediately after overstrain. In the first test the specimen was given a permanent elongation of about 1 per cent. After release of the tensile load it was subjected to compression, but showed a loss of elasticity as seen in the stress-diagram. When compressed to its original length and reloaded in tension it showed loss of elasticity in tension. After being again overstrained in tension about

1 per cent it once more became elastic, but lost its elasticity when elongated 18 per cent and reloaded in compression. Similar results for steel also are reported in *Tests of Metals*, 1899.

**672. The toughness of wrought iron** under repeated shocks has lead many engineers to specify it in preference to soft steel for certain types of shafting. It is also claimed that wrought-iron studs for cylinder heads are tougher than those made of steel and that wrought-iron pipe buckles less often than steel in driving. Whatever evidence there may be of the superiority of individual parts, it does not seem fair, in view of variations in the qualities of both of the materials, to make such claims for class superiority.

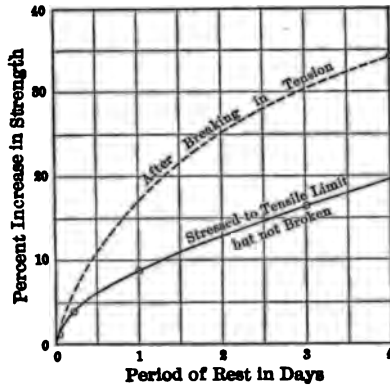


FIG. 5.—Increase in the Tensile Strength of Wrought Iron after having been Stressed to the Tensile Limit and Allowed to Rest. Points represent 5 to 15 results each. (*Report U. S. Test Board*, 1881, Vol. I, p. 111 and 115).

Experimental evidence seems to show that soft steel of good quality has more capacity for absorbing the energy of a single blow. On the other hand, basing judgment upon results of impact tests on slotted bars, it appears that good wrought when overstrained retains as high resistance to impact as soft steel similarly overstrained.

**673. The Strength of Wrought-iron Chains.**—The hundreds of tests on wrought-iron chains given in the *Report of the U. S. Test Board*, 1881, Vol. 1, show that the ultimate strength of

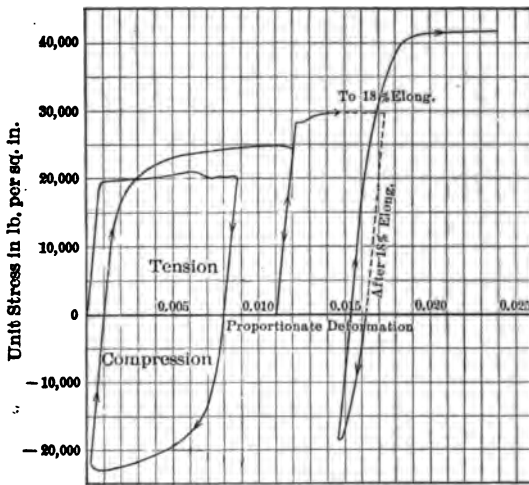


FIG. 6.—Showing that a Small Permanent Set in Wrought Iron either in Tension or Compression Greatly Reduces the Elastic Limit in the Opposite Stress, when that Stress is Immediately Applied. (Gray, in *The Digest of Physical Tests*, Vol. 1, p. 232, 1896.)

chains may be taken at 1.6, that of the iron from which the links are

made. It also appears from these tests that open links are somewhat stronger than studded links, though the open-link chains take a permanent set earlier than the studded links. It is thought, however, that open-link cables would foul more readily than the studded cables. The elastic properties of open-link chains made of 1-in. and  $\frac{3}{4}$ -in. iron are shown in Fig. 7, where the tests have been carried to the proof-load only, this being such as to give to the chains a permanent set of about 2 per cent of their length. They now become perfectly elastic to 20,000 and 15,000 lb. respectively, and are also about five times stiffer, or more rigid, than they were at first. All chains are improved by this treatment, while it also discovers any very poor welds the chain may have.

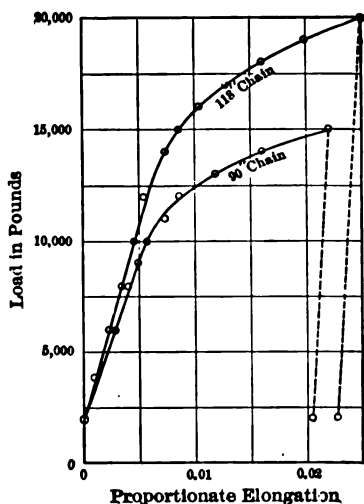


FIG. 7.—Proof Tests of Wrought-iron Chains with Open Links. The 118-in. chain was made of 1-in. iron, the 90-in. chain of  $\frac{3}{4}$ -in. iron. (*Tests of Metals*, 1894.)

have arrived at the following stresses in chain links:

for open links,

$$S_t = \frac{2P}{A},$$

for stud links,

$$S_t = \frac{1.6P}{A},$$

where  $S_t$  is the maximum unit tensile stress,  $P$  the load on the chain, and  $A$  the area of the stock. For either of the above chains the compressive stress on the inner fibers at the link ends is approximately double the maximum tensile stress. However, since the length of link under compression is very short and since additional security is affected at these points by the pinching action of the adjoining links, it appears practical to base the design upon the tensile strength of the link. If 15,000 lb. per square inch is considered a safe value for tensile stress in chains, then the above formulas reduce to  $P=6000 d^2$  for open links and  $P=7500 d^2$  for stud links, where  $d$  is the diameter of the iron.

**674. The Welding of Wrought Iron.**—One of the most valuable properties of wrought iron is the facility with which portions of

\* Bull. No. 18, Engr. Expt. Sta. Univ. of Ill.

it may be united by squeezing or hammering while at a high heat. This characteristic renders the iron very useful to the smith. The property is due to the high plasticity of the iron throughout a considerable range of temperature extending below a white heat. However, if the iron parts are heated to such temperatures in the presence of much air their surfaces soon become coated with a film of iron oxide. Consequently it is difficult to bring the welding surfaces into perfect contact. To avoid imperfect cohesion it is necessary that the central portions of the surfaces in the weld be first brought together so that the oxide forming at the joint may be expelled outward and not entrapped as the seam is closed under the hammer. The evil effects of the oxide may be greatly reduced by using borax or other flux which acts as a solvent of the oxide and renders it more easy to expel from the joint. Moreover, by maintaining a thick fire in which most of the oxygen is combined with carbon, or by heating the parts in a muffle it is possible to considerably reduce the amount of slag formed. One of the advantages of electric welding lies in the fact that no air blast is employed, and by having the parts in contact during the



FIG. 8.—Common Types of Welds.

heating period, air is largely excluded from the welding surfaces and hence little oxide is there formed.

Fig. 8 shows the common types of welds and the shapes of the parts prior to welding. After the parts have been shaped and upset as required by the work in hand, they are reheated, if necessary, and then rapidly hammered until the metal is below a red heat. This hot work reduces the grain size and renders the metal at the weld more ductile and tough. Inasmuch as there is likely to be overheated metal on either side of the joint which has not been properly worked during the welding, it is best, wherever possible, to anneal welds and thus secure uniformity in structure and properties of the metal in the vicinity of the joint.

On account of the difficulties encountered in welding, the strength of welded joints is likely to vary from 30 to 100 per cent of the strength of the parts joined. With the most careful hand welding Kirkaldy reports the following average efficiencies: For the strengths of joints in round tie-rods  $1\frac{1}{2}$  to  $3\frac{1}{8}$  in. in diameter 60 per cent; for flat plates  $2\frac{1}{2}$  to 6-in. wide and  $\frac{1}{4}$  to 1-in. thick 71 per cent; for chain-

and cast iron; the latter being non-malleable at any range of temperature.

**678. The Physical Characteristics of Ferrite and Cementite.**—Again referring to regions IX and XII of the Roozeboom Diagram we note that slowly cooled steels and cast iron consist of pure iron (ferrite) and iron carbide (cementite), a portion or all of the ferrite and cementite always being intimately associated in the eutectoid, pearlite. Ferrite is a relatively soft, ductile and malleable metal of low elastic ratio. Cementite, on the contrary, is extremely hard, brittle, non-malleable at any temperature, and has a high elastic limit equaling its tensile strength. It is to be expected, therefore, that the relative properties of these two constituents, together with the nature of the association and degree of aggregation, will determine the physical properties of steels and cast irons. In fact the strengths of normal carbon steels of less than eutectoid composition are in direct ratio to the percentage of pearlite, whereas in the hypereutectoid steels the strengths diminish slightly as the excess cementite increases.

**679. The essential relations between carbon content and mechanical properties** for the carbon-iron alloys containing 4 per cent or less of carbon are shown in Fig. 1. From the figure it will be observed that

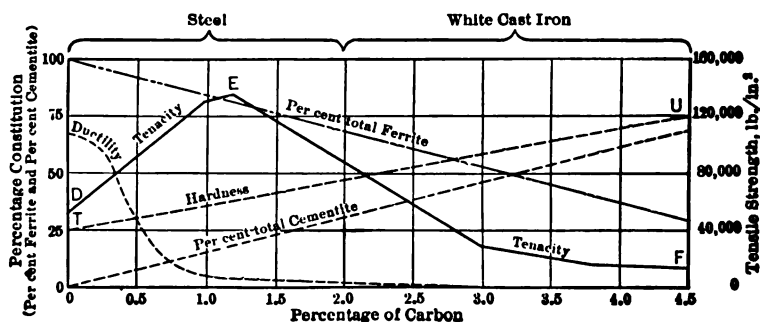


FIG. 1.—Theoretical Relations between Mechanical Properties and Carbon Content in Iron-Carbon Alloys. (Howe.)

the tensile strength increases to a maximum when the metal is approximately of eutectoid composition. This is probably due to the relatively intimate mixture and fine state of aggregation of the constituents of the pearlite. With a decrease or increase of carbon the ferrite or cementite, respectively, becomes an excess substance forming a grain network which has a weakening influence on the metal.

The relation of hardness to carbon content is dependent upon the kind of hardness. In Fig. 1 hardness is shown as varying directly with the increase in cementite. As measured by the Brinell ball method the resistance to indentation increases much more rapidly for

carbon contents up to the eutectoid ratio (0.90 per cent C.) than for the higher percentages. Thus the Brinell number for soft iron lies between 75 and 80, for eutectoid steel it is approximately 250, and for

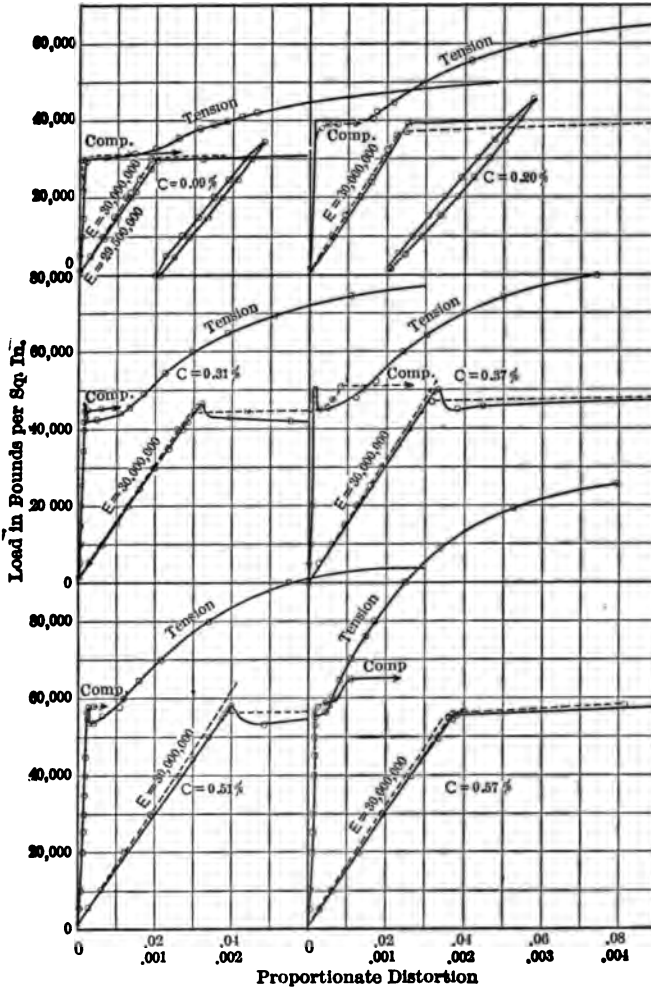


FIG. 2.—Compression and Tension Stress-diagrams of Steel Bars of Varying Percentages of Carbon.

Compression specimens 12 in. long and 1 in. in diam. ( $\frac{l}{r} = 33$ ) with flat ends. Tension specimens same diam. with a gauged length of 30 in. All specimens turned from open-hearth steel bars  $1\frac{1}{4}$  in. in diam. (*Tests of Metals*, 1886 and 1887.)

white cast iron it ranges from 350 to 500. Brinell's tests on carbon teels (Fig. 21) show that mean rate of increase in ball hardness is 19 Brinell scale) per 0.10 per cent increase in carbon until the carbon reaches 1.0 per cent. Again from tests with various abrasive powders



F. Robin found that the wearing resistance of annealed or normal carbon steels was greatest when the carbon content was about 0.40 per cent. It is apparent, therefore, that the relation of hardness to carbon content varies greatly for different kinds of hardness.

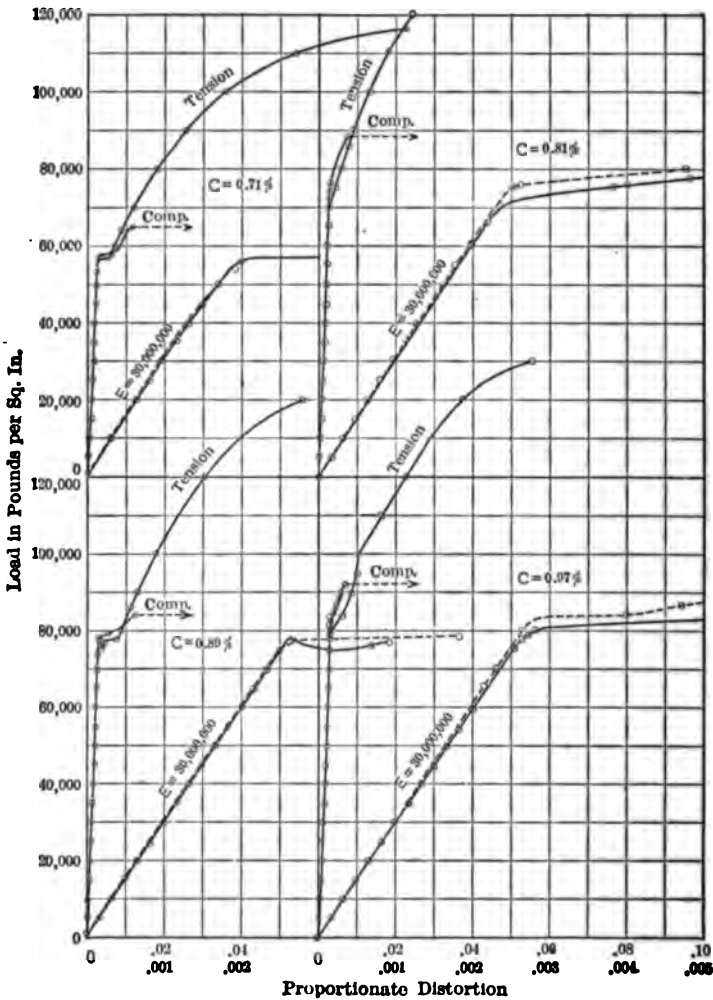


FIG. 3.—Supplementary to Fig. 2.

**680. Influence of Carbon on Strength of Steel.**—A study of the data shown in Fig. 2 to 5 indicates that the tensile strength of hot rolled steel bars varies from 55,000 lb. per square inch for 0.1 per cent carbon to 150,000 lb. per square inch for 1.0 per cent carbon. The strength of the hot rolled bars, Fig. 4, is a maximum between 1.0 and

1.2 per cent carbon; whereas, among the annealed bars, Fig. 5, those of eutectoid composition are the strongest.

If the strength given in Figs. 2 and 3 are also plotted on Fig. 4 it will be found that there is approximately a linear relation between the tensile strength ( $S_t$ ) and the per cent carbon ( $C$ ) for carbon content less than 1.00 per cent. This relation may be expressed by

$$S_t = 45,000 + 115,000 C. \quad (1)$$

For annealed steels with low content of impurities Sauveur \* proposes the following simple formulas:

$$S_t = 50,000 + 90,000 C \quad (C < 0.83 \text{ per cent}) \quad (2)$$

$$S_t = 142,000 - 20,600 C \quad (C > 0.83 \text{ per cent}) \quad (3)$$

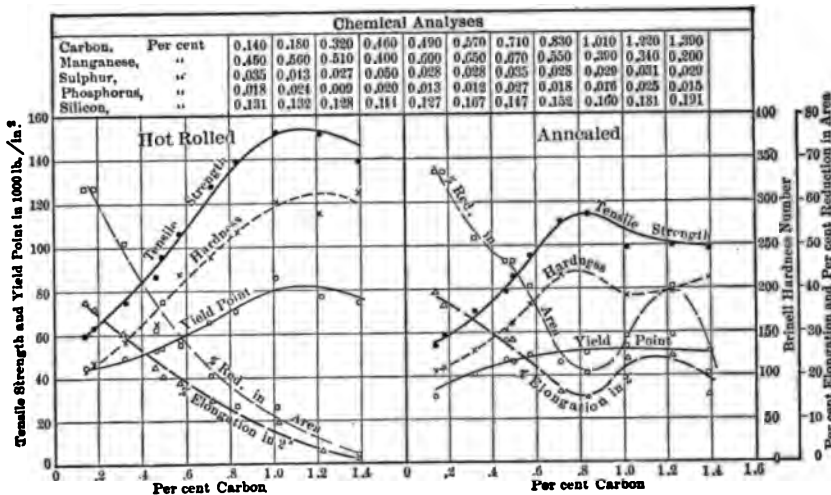


FIG. 4.

FIG. 5.

FIG. 4.—The Influence of Carbon on the Mechanical Properties of Hot-Rolled Steels. (Nead.) Diameter of specimens 0.505 in.; gage length = 2 in.

FIG. 5.—The Influence of Carbon on the Mechanical Properties of Annealed Steels (Nead). Annealing temperatures were those recommended by A. S. T. M., see Fig. 10.

Formula 3 gives results about 10,000 lb. per square inch higher than the data for annealed steels in Fig. 5.

From an extensive series of tests on acid and basic open-hearth steels Campbell † derived the following more complex formulas for the tensile strength ( $S_t$ ) of steel.

$$\text{For acid open-hearth steel, } S_t = 40,000 + 1000C + 1000P + xMn + R. \quad (4)$$

$$\text{For basic open-hearth steel, } S_t = 41,500 + 770C + 1000P + yMn + R. \quad (5)$$

In these equations  $C$ ,  $P$ , and  $Mn$  represent the number of points ‡ of carbon (by combus-

\* *The Metallurgy and Heat Treatment of Iron and Steel*, p. 138.

† *The Manufacture and Properties of Iron and Steel*, p. 391.

‡ A point is equivalent to 0.01 per cent.

tion), phosphorus, and manganese, respectively, and  $R$  is a variable depending on the heat treatment. For rolled steels finished at the ordinary temperatures  $R=0$ . If carbon is determined by the color method the coefficients for  $C$  in Eq. (4) and (5) should be 1140 and 820, respectively. Values of the coefficient  $x$ , for manganese over 40 points and carbon under 60 points, are given by  $x=8C$ ; values of  $y$ , for manganese over 30 points and carbon under 40 points, are given by  $y=4C+90$ , where  $C$  is the number of points carbon as before.

The *elastic limit* and the yield point of steel in tension, like the ultimate strength, increase with the carbon content but at a lower rate.

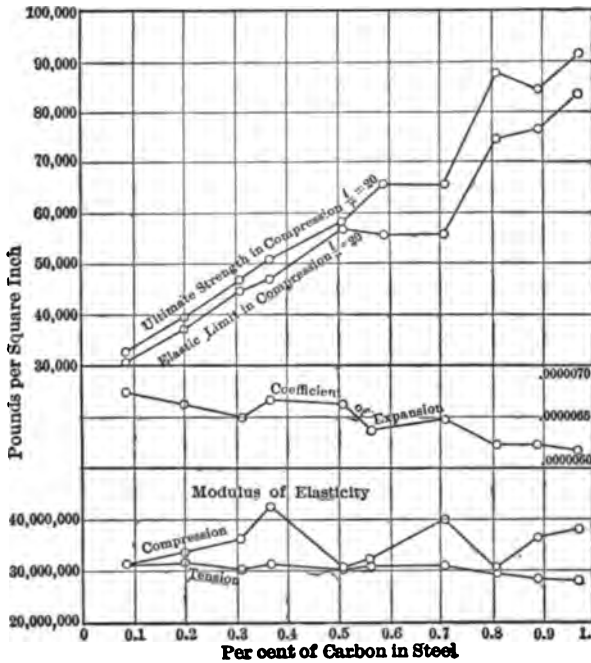


FIG. 6.—Variation of Compressive Strength, Coefficient of Expansion and Modulus of Elasticity with Carbon Content. (*Tests of Metals*, 1886.)

Values for modulus in compression appear to be in error.

The elastic ratio usually runs between 0.60 and 0.70 for the medium- and low-carbon steels and between 0.50 and 0.60 for the high-carbon varieties. Thus from the results in Figs. 2, 3 and 4, the following approximate relation  $S_y=30,000+48,000 C$  is obtained. Here  $S_y$  is the unit stress at yield point and  $C$  the per cent carbon ( $< 1$ ).

The yield point in high-carbon steels is much less pronounced than in low- or medium-carbon steels as the diagrams of Figs. 2, 3 and 9, indicate.

The *ultimate compressive strength* of steel in very short prisms is not well defined. In members with  $\frac{1}{r}$  above 100, column action (Art. 21) is introduced, but in fairly short prisms with  $\frac{1}{r}$  between 20 and 50 the compressive strength is practically equal to the apparent elastic limit, see Figs. 2, 3 and 6. From Fig. 6 it appears that the compressive strength increases directly with the carbon content up to 1 per cent carbon,

the relation for these data being expressed by  $S_c = 37,000 + 55,000 C$ . Results of certain tests by Marshall, given in Table 1, show that the elastic limit in tension and compression are practically identical for all sizes of bar and also that the ultimate compressive strength of bars with  $\frac{l}{d} = 12$  is the elastic limit. They also show the marked decrease in elastic limit and ductility for the large sizes of bar, all being rolled from the same billet.

TABLE 1.—MILD STEEL IN TENSION AND COMPRESSION

Comparison of Tensile and Compressive Results with Results of Tests on Short Columns of Round and Square Bars from  $\frac{3}{4}$  in. to  $2\frac{1}{2}$  in. in Diameter, all Rolled from one Blow of Bessemer Steel. Elongations measured on a length equal to ten diameters, by means of an extensometer similar to Fig. 37, Chap. II. (From Marshall's Experiments, *Trans. Am. Soc. C. E.*, Vol. 17, Tables 1 and 2.)

Size of Specimen.	ELASTIC LIMIT IN POUNDS PER SQUARE INCH.		ULTIMATE STRENGTH IN POUNDS PER SQUARE INCH.		ELONGATION.		Reduction of Area. Percentage.
	In Tension.	In Compr. for $\frac{l}{d} = 2$ .	In Tension.	In Compr. for $\frac{l}{d} = 12$ .	Length of Specimen.	Percentage of Elongation.	
$\frac{3}{4}$	45,181	45,000	68,711	44,970	8	26.4	45.3
1	43,880	45,355	68,240	43,540	10	25.6	39.3
$1\frac{1}{4}$	40,903	42,880	67,506	40,455	12	26.4	43.0
$1\frac{1}{2}$	39,795	42,015	66,598	40,150	15	25.4	39.3
$1\frac{3}{4}$	39,105	41,225	66,366	39,700	18	24.3	33.3
2	38,207	39,176	65,663	40,300	20	23.9	27.8
$2\frac{1}{4}$	37,655	36,542	65,460	38,030	22	13.7	17.2
$2\frac{1}{2}$	36,100	36,840	.....	35,650	25	10.2	
Means	40,103	41,129	66,935	40,350	.....	21.9	35.0

The *shearing strength* of steel also increases with the carbon content, as can be seen from the results in Table 2. The test-pieces used in obtaining these results were subjected to double shear, due care being taken to avoid bending as far as possible.

From these data it appears that the ratio of the shearing strength to the tensile strength is approximately 0.80 for medium- and low-carbon steels, but decreases as the carbon content increases, being in the vicinity of 0.60 for high-carbon steels.

Fig. 7 shows the influence of carbon on the torsional strength and torsional ductility of  $\frac{5}{8}$ -in. round steel specimens of variable carbon content. The ratio of the computed twisting strength to the tensile strength for these steels varied from 1.1 for the low-carbon steels

TABLE 2.—SHEARING RESISTANCE OF STEEL

Kind of Steel.*	Per Cent. Carbon.	Number of Tests Averaged.	TENSION TEST.		Shearing Strength, Pounds per Sq. In.	Ratio of Shearing to Tensile Strength.
			Elastic Limit, Pounds per Sq. In.	Ultimate Pounds per Sq. in.		
Landore Siemens . . . . .		2	37,500	57,000	47,500	0.829
" " . . . . .		2	40,000	63,500	51,000	0.800
" " . . . . .		3	37,000	64,000	52,000	0.811
" " . . . . .		6	40,600	69,000	56,000	0.807
Weardale Bessemer . . . . .		6	44,000	71,000	51,000	0.715
Bessemer . . . . .		4	51,500	78,000	64,000	0.823
" . . . . .		4	62,000	82,000	59,000	0.721
Crucible . . . . .		2	69,500	116,000	74,000	0.632
Bessemer . . . . .		2	70,000	118,000	79,000	0.670
Swedish Crucible . . . . .	0.12	Average of a	55,800	55,800	41,500	0.740
	0.48		94,400	94,400	64,500	0.680
	0.71	Large Number	126,100	126,100	82,000	0.650
	0.77		137,300	137,300	85,400	0.620

\* Tests on Swedish crucible steels were reported by E. G. Isod in *Proc. Inst. Mech. Engrs.*, 1906-1, p. 9; others by A. B. W. Kennedy in *Proc. Inst. Mech. Engrs.* 1885, p. 262.

to 0.9 for the high-carbon steels. Since the true shearing strength is three-quarters of the modulus of rupture in torsion in, according to

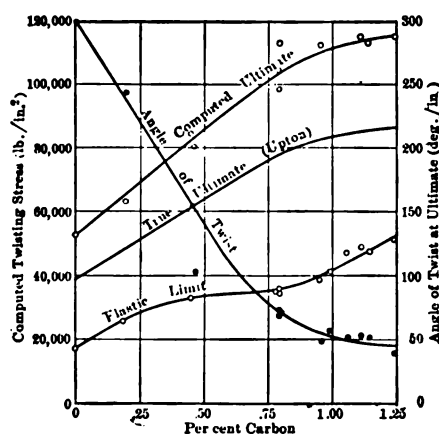


FIG. 7.—The Effect of Carbon on the Torsional Properties of Steel. Each point represents 3 tests on  $\frac{3}{8}$ -in. round specimens. (*Tests of Metals*, 1881, p. 287.)

This is shown in Figs. 2 and 3. Table 3 from Marshall's carefully conducted experiments\* also furnishes comparisons of the moduli of elasticity for steels varying in carbon content.

\* Reported in *Trans. Am. Soc. Civ. Engrs.*, Vol. 17, p. 62.

Upton's theory, the ratio of the true shearing strength in torsion to the tensile strength varied from 0.82 for the low-carbon steels to 0.67 for the high-carbon varieties. These ratios are in good agreement with the values computed from the transverse shear results of Table 2. The ratio of the elastic limit in torsion to the elastic limit in tension was more nearly constant, being approximately two-thirds for all carbon contents.

681. The modulus of elasticity of steel is nearly the same for tension and compression, and for either stress it is practically independent of the carbon content.

TABLE 3.—MODULI OF ELASTICITY OF STEEL ON FIRST LOADINGS, WITH VARYING PERCENTAGES OF CARBON, ONE SPECIMEN FROM EACH HEAT. (MARSHALL)

Number of Heats and Tests.	Average Percentage of Carbon.	MODULI OF ELASTICITY $E$ , IN POUNDS PER SQUARE INCH.			Kind of Steel.
		Lowest Value.	Highest Value.	Average Value.	
33	.09	28,750,000	31,540,000	29,924,000	Bessemer
8	.11	29,210,000	30,670,000	30,020,000	Open-hearth
107	.24	28,310,000	31,180,000	29,996,000	"
89	.34	28,140,000	30,910,000	29,672,000	Bessemer
25	.72	28,680,000	30,860,000	29,919,000	Open-hearth
Weighted mean value =				29,866,000	

TABLE 4.—COMPARISON OF MODULI OF ELASTICITY IN TENSION AND COMPRESSION. (MARSHALL)

All results given in one-thousand-pound units, identical material.

STEEL.—Tensile Strength less than 100,000 Pounds per Square Inch.					SPRING-STEEL.—Tensile Strength 144,000 Pounds per Square Inch.				
Size of Bar, In.	TENSION.		COMPRESSION.		Size of Bar, In.	TENSION.		COMPRESSION.	
	$E_1$ First Loading.	$E_2$ Second Loading.	$E_1$ First Loading.	$E_2$ Second Loading.		$E_1$ First Loading.	$E_2$ Second Loading.	$E_1$ First Loading.	$E_2$ Second Loading.
1 rd.	30,190	34,420	29,450	29,740	1 rd.	29,480	29,760	28,880	29,300
$\frac{1}{16}$ sq.	29,850	29,850	28,070	29,010	1 rd.	29,390	29,580	28,880	29,200
1 rd.	29,280	29,500	28,780	29,420	$\frac{1}{16}$ sq.	28,880	29,420	29,090	29,220
$\frac{1}{4}$ rd.	29,830	29,150	28,580	29,420	$\frac{1}{16}$ sq.	29,200	29,200	29,090	29,350
$\frac{1}{16}$ sq.	29,420	29,640	28,380	28,670	Mean	29,237	29,490	28,985	29,267
1 rd.	29,550	29,630	28,680	28,830					
1 rd.	29,240	29,960	30,070	30,490					
1 rd.	29,400	30,420	28,980	29,790					
1 rd.	30,000	30,370	29,260	29,810					
Mean	29,529	30,371	28,884	29,464					

Table 4 shows that the modulus of elasticity of steel on second loading is likely to be slightly higher than on first loading due to very small set, although the primitive loading does not exceed the limit of proportionality. If the primitive loading exceeds the proportional limit, the modulus of elasticity is lowered, in some cases from 10 to 20 per cent. From a consideration of the available experimental data, it appears that the modulus of elasticity of steel in tension or compression

generally lies between 28,500,000 and 31,000,000 a mean value being about 29,500,000 lb. per square inch.

A very large number of tests by Brinell on the modulus of elasticity of steels show also that heat treatment has little effect on the modulus of elasticity of carbon steels regardless of their carbon content. Annealed steels and quenched steels gave moduli averaging from 3 to 4 per cent higher than the bars received from the rolls.\*

The modulus of elasticity in shear (often called the modulus of rigidity) is determined from torsion tests. Bauschinger reported tests † on Bessemer steels varying in carbon content from 0.19 to 0.96 per cent and on open-hearth steels of five degrees of hardness. For the Bessemer steels the shearing modulus varied from 11,900,000 to 12,700,000 lb. per square inch, and for the open-hearth varieties it ranged between 11,500,000 and 12,250,000 lb. per square inch. Platt and Hayward ‡ also reported values of the shearing modulus of elasticity for high- and low-carbon steels,—Bessemer, crucible and open-hearth varieties being represented. Their results ran from 12,350,000 to 14,000,000 lb. per square inch. For low- and medium-carbon steels an average value of the shearing modulus is 12,000,000 lb. per square inch.

**682. Influence of Carbon on Ductility.**—The ductility of steel decreases markedly as the carbon content increases (Figs. 4 and 5). Since ductility is also much influenced by variations in heat treatment and by the gauge length (Art. 711 and 106) it is not possible to give an accurate equation between ductility and carbon content. The following equations between per cent elongation ( $e$ ) and per cent carbon ( $C$ ), or strength ( $S_t$ ), are examples of some of the attempts to express such relationships.

$$e = \frac{3}{C^2 + 0.1}, \text{ by Howe for elongation in 8 in. . . . . (1)}$$

$$e = 32 - 30C, \text{ by Upton for specimens having diameter of 0.8 in. and 8 in. gauge length. (2)}$$

$$e = 50 - 48C, \text{ by Upton for } \frac{1}{2}\text{-in. standard test-piece . . . . . (3)}$$

$$e = \frac{1,800,000}{S_t - 10,000} - 10, \text{ by J. B. Johnson for elongation in 8 in. . . . . (4)}$$

$$e = \frac{1,500,000}{S_t}, \text{ by a Committee of Am. Soc. Civ. Engr . . . . . (5)}$$

The range of values for  $C$  in Eq. (1) to (3) is from 0 to 1 per cent. Eq. (1) gives values about 5 per cent to low for medium-carbon steels. Equations of the same type as 5 are common in specifications. For the high-carbon steels, however, this equation calls for too great elongation. Therefore, the constant is reduced in specifications for such steels.

\* *Jour. Ir. and St. Inst.*, Vol. 60, p. 234.

† See Unwin's *Materials of Construction*, p. 241.

‡ *Proc. Inst. Civ. Engr.*, Vol. 90, p. 409.

Fig. 8 shows the elongation field as worked out from a table in Howe's *Metallurgy of Steel*, which gives values of greatest and least elongations for different steels. It is plain from the figure that Eq. (4) has a much wider range of application than Eq. (5) or others of that type (5).

The per cent reduction in area ( $R$ ) also decreases as the carbon content increases. In Fig. 4 the relation may be approximately expressed by

$$R = \frac{13}{C^2 + 0.18}.$$

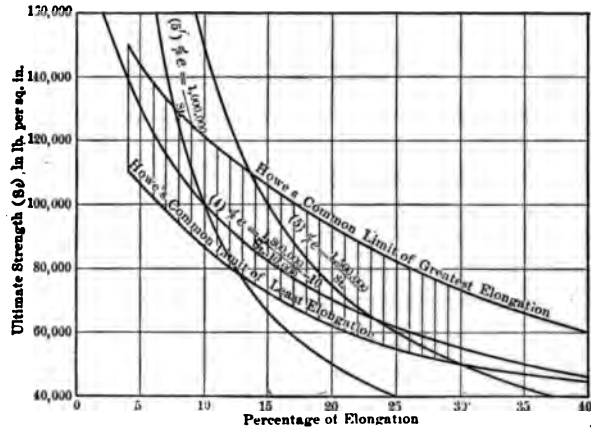


FIG. 8.—Relations of Elongation to Strength for Steels.

### 683. Changes in

**the Shape of the Stress diagram.**—The effects of carbon on the tensile properties of steel are well shown by the changes in the shape of the stress-diagram. Thus, in Fig. 9 the increases in the elastic limit and ultimate strength and the accompanying decreases in the elongation cause the diagrams to increase in height and decrease in width. The horizontal portion of the curve for low-carbon steel, which follows the drop in the load after the yield point has been passed, disappears in the diagrams for the high-carbon specimens. The downward slope of the portion of the stress-deformation curve beyond the maximum stress becomes less pronounced in the high-carbon steels and disappears in the diagrams of the very hard steels which break without necking. Furthermore, the area of the diagram, representing the energy of rupture, decreases as the carbon increases (Fig. 9).

**684. The resistance of steel to heavy shocks or blows decreases as the carbon content increases.** The decrease in shock resistance is most rapid as the carbon is increased from 0.15 to 0.40 per cent. Fig. 33 shows some of Brinell's impact results which emphasize this conclusion. The figure also shows that the energy of rupture computed from tensile tests is a poor measure of resistance to impact. Similar evidence showing discrepancies between the energies of rupture in tension and impact is furnished by data of the Alloys Research Committee\* as compiled by Howe in the discussion of a paper on impact testing before the Institute of Mining Engineers.†

\* *Proc. Inst. of Mech. Engr.*, 1904-1, p. 160.

† *Trans. Am. Inst. of Min. Engr.*, Vol. 53, p. 218.



Although it appears that the soft and medium-carbon steels as rolled, or in the annealed state, have much superior resistance to impact than the high-carbon steels, we must not infer from this conclusion that the low-carbon steels are best suited to withstand repeated stress or a succession of light blows. Under such loadings, the harder steels with a higher elastic limit have proven more satisfactory. (See Art. 828.)

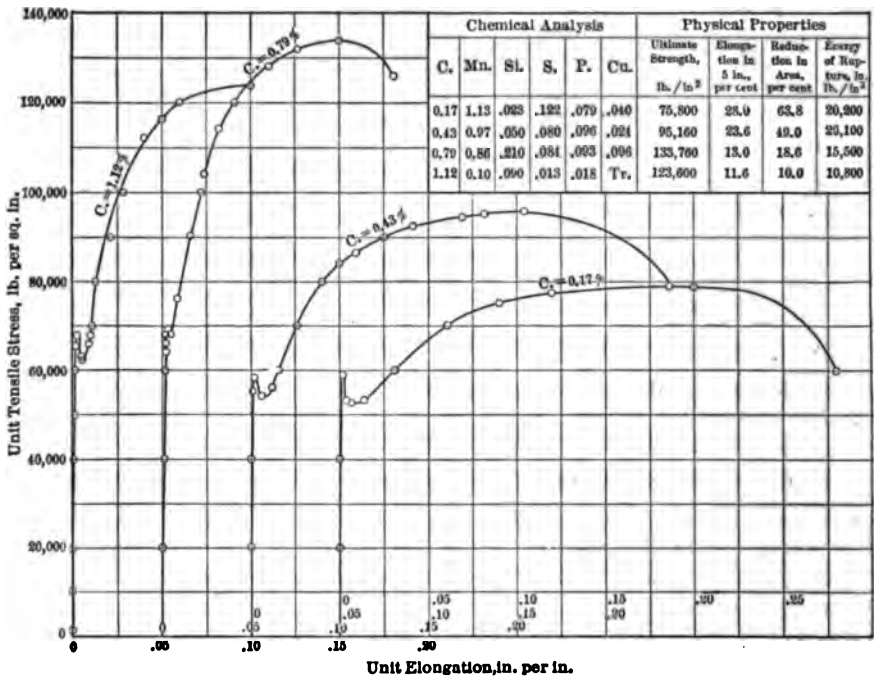


FIG. 9.—Tension Stress-diagrams of Carbon Steel Bars Used for Endurance Tests of Shafting. (*Tests of Metals*, 1889, p. 389; 1891, p. 603.)

**685. The Range in Composition of Structural Steels.**—Practically all steels used in engineering construction in the normal or annealed condition are of less than eutectoid proportion (0.9 per cent carbon). The carbon content is the result of experience whereby the proper combination of strength, elasticity, hardness and workability have been obtained to fit the material for the particular service. Increased tenacity and elasticity are gained at the sacrifice of ductility and softness. In structural steels, boiler plate, and the like, where some increase of strength is desirable, provided there is not too great loss of ductility and softness, a carbon content of about 0.20 per cent is customary. The tensile strength of the iron is thereby increased by about 15,000 lb. per square inch (from 45,000 to 60,000) without

material loss of the ductility and softness so essential to the requirements for manufacture and service. In steel rails, there is little fabrication of the finished product and the stresses are heavy shocks at intermittent periods. Hardness is essential for wear, and high strength and elasticity are necessary to enable the steel to withstand the load and recover its alignment; yet ductility cannot be sacrificed to the extent of dangerous brittleness. The carbon content of rails varies from 0.50 to 0.70 per cent, the larger amounts having been introduced in special cases in recent years, particularly in heavy sections and here the steel has the minimum of phosphorus, sulphur, or other associated detrimental elements, which lower ductility without the compensating advantage of adding much strength.

The following table shows classifications, approximate composition and mechanical properties of some of the more important steels used in construction. Carbon contents for tool steels may be found in Art. 709.

CLASSIFICATION BASED ON			Per Cent Carbon.	Tensile Strength (lb./in. <sup>2</sup> ).	Percent Elongation in	
Usage.	Hardness.	Manufacture.			8 in.	2 in.
rivet.....	Extra soft	O. H.	0.08-0.15	45- 55,000	30	
tubing and Pressed Metal	Extra soft	O. H.	0.08-0.15	45- 55,000	30	
new stock.....	Mild or Soft	O. H.	0.10-0.20	55- 65,000	25	
oil plate.....		Bess.				
		O. H.	0.10-0.20	55- 65,000	25	
		O. H.				
structural.....	Medium	Bess.	0.10-0.20	55- 65,000		
structural.....		O. H.	0.20-0.35	65- 75,000	22	
machine.....	Medium	O. H.	0.20-0.30	60- 70,000	23	
rail.....	Hard	O. H.	0.40-0.75	85-125,000	..	10-15
		Bess.				
re.....	Hard	O. H.	0.50-0.80	110-130,000	..	10

#### EFFECTS OF PRINCIPAL IMPURITIES ON STEEL \*

**686. General Effects.**—As we have seen in Chapter XVIII, it is not feasible under present practice to remove impurities entirely in making either iron or steel. Therefore, the final product always contains, besides iron and carbon, small percentages of the metallic impurities,—silicon, manganese, sulphur, and phosphorus,—together with lesser amounts of the oxides of silica, manganese and iron, silicates of manganese and iron, and some occluded gases. Occasionally very

\* For an extended discussion of the effects of impurities on steel the reader should consult the excellent paper and bibliography compiled by J. E. Stead, See *Jour. Ir. and St. Inst.*, Vol. 94, p. 5-136.

small percentages of copper and arsenic are also present. In well-made steel the total amount of these impurities generally ranges between 0.2 and 1.0 per cent and their resultant effect on the constitution of steel is often very small. Of the common impurities, phosphorus and sulphur are injurious elements present in the ore which cannot be eliminated in the process of manufacture, whereas most of the silicon and manganese are introduced to improve the metal. The non-metallic impurities are objectionable substances which find their way into the steel during the process of refining.

**687. Effects of Silicon.**—Silicon probably in the form of iron-silicide, forms solid solutions with iron in all proportions up to 20 per cent. It is often added to molten metal to remove oxygen and diminish blowholes. In the carbon steels silicon rarely exceeds one-half of one per cent and in structural steels it is generally under a quarter of a per cent. With such small proportions of silicon the microscope reveals no peculiarities in constitution. Silicon up to 1.75 per cent appears to increase both ultimate strength and elastic limit without decreasing ductility.\* Brinell's researches show that silicon increases the hardness of steel. For silicon varying between 0.10 and 0.20 per cent the increase in hardness was approximately 6.4 (Brinell scale) per 0.1 per cent increase in silicon. Silicon is, therefore, about one-third as effective as carbon in increasing hardness.

On account of the marked tendency of silicon to prevent solution of carbon in iron (Art. 662) it is very necessary to avoid prolonged heating at high temperatures in treating steels having high silicon and carbon contents. Instances have been recorded where steel castings have been ruined by soaking for a long time at a temperature considerably above the recalescence point, the combined carbon being thus transformed into graphite and the casting thereby greatly embrittled. However, with normal percentage of silicon and good heat treatment graphite is not present in steel.

**688. Effects of Phosphorus.**—When present in the low proportions common to steel, phosphorus exists in a solid solution of iron-phosphide ( $\text{Fe}_3\text{P}$ ) and iron. In this form it permeates both the free ferrite and that composing the pearlite (Sauveur). By some it is considered to promote enlargement of the grains and thus produce brittleness. Howe maintains that the presence of phosphorus in ferrite makes the ferrite more mobile when it is slowly cooling through the transformation range. This mobility results in the banding of the ferrite into thin rods or layers which, upon etching, are revealed as light-colored lines, called ghost lines. Such formations, of course, render the metal less homogeneous.

\* The properties of silicon steels are discussed in Art. 734.

In wrought iron much of the phosphorus content is held in the slag fibers in the form of iron phosphate. As a constituent of the slag its effect is probably less detrimental than as a phosphide in the ferrite.

Although the ductility of low-carbon steel may be slightly decreased by the presence of 0.3 to 0.5 per cent phosphorus, the yield point, ultimate strength and hardness are increased. Resistance to shock is, however, markedly reduced by such high percentages of phosphorus. Tests by D'Amico on low-carbon steels show that toughness is adversely affected (i.e., the metal is rendered cold short) by 0.1 per cent phosphorus and very much decreased by 0.2 per cent of this element. The evil effect on toughness appears to be more pronounced in high-carbon than in low-carbon steels.

Although it is very probable that many failures have been erroneously ascribed to a high phosphorus content which, if all facts were known, were traceable to other causes, yet a due regard for the tendency to produce cold shortness requires that phosphorus be kept low in steel. At present the maximum limits are: for inferior grades of structural steel 0.1, for best grades of structural steel 0.05, and for tool steels 0.02 per cent.

**689. Effects of Sulphur.**—Sulphur readily combines with iron to form iron sulphide ( $\text{FeS}$ ) which, when present in iron or steel, has a tendency to segregate and form brittle networks at the grain boundaries. On account of its low melting-point, iron sulphide causes a lack of cohesion between adjacent grains of the metal when it is heated above a red heat. Such brittleness at high temperatures is termed red shortness. Since red shortness makes steel or iron hard to roll or forge, it is a serious defect in the metal from the standpoint of the manufacturer. Manganese sulphide has a much higher-melting point than iron sulphide and does not render ferrous metals red short. Therefore, inasmuch as manganese has a very powerful affinity for sulphur, it is possible to relieve red shortness by adding sufficient manganese to the molten metal to combine with the sulphur. Theoretically the ratio of manganese to sulphur should be 1.7 to 1 in order to form manganese sulphide ( $\text{MnS}$ ) and completely satisfy sulphur. Levy contends, however, that even if manganese is present in sufficient quantities to form the sulphide some iron sulphide will still remain and will be found mixed with the manganese sulphide. Since manganese sulphide also segregates and forms brittle masses, more or less rounded in castings and elongated in mechanically worked pieces, it appears that either sulphide causes a lack of homogeneity.

If sufficient manganese is present to prevent red shortness there is little evidence that sulphur in quantities less than 0.15 per cent exercises any appreciable effect on the mechanical properties of structural

steel.\* For screw-stock a high sulphur content (0.10 to 0.15 per cent) is commonly specified, since chips of such metal crumble without curling and the stock threads nicely. There appears to be a feeling, even though there is little direct evidence to support it, that percentages of sulphur too small to produce red shortness in rolling may develop invisible flaws in the finished metal. Specifications for steel, therefore, stringently limit sulphur to practically the same amounts as phosphorus.

**690. Effects of Manganese.**—Manganese is one of the main elements of the recarburizers used in manufacturing steel. Through its strong affinity for oxygen and sulphur manganese acts as a cleanser of the molten metal by withdrawing much of these undesirable impurities into the slag. When more manganese is present than required to satisfy sulphur and oxygen the excess manganese forms the carbide,  $Mn_3C$ , which is associated with cementite. If present in this form it acts as a hardener. In carbon steels, the manganese content is generally under 1 per cent and ordinarily runs about 0.3 to 0.6 per cent. Brinell's tests show that manganese, when under 1.0 per cent, hardens steel slightly, the average increase in hardness due to an increase of 0.1 manganese being about 4.0 on the Brinell scale. Howe † claims that a maximum fineness of grain together with a minimum of injury in hardening can be secured for carbon steels by maintaining manganese high, say around 1.25 per cent.

In high-carbon alloys manganese increases the solubility of carbon in iron and hinders the precipitation of graphite in cooling.

**691. Copper** up to 0.5 per cent appears to have little if any effect on the mechanical properties of steel. A small percentage of it is said to increase resistance to corrosion. In most steels the proportion of copper is negligible.

**692. Arsenic** is occasionally found in very small proportions in steels made in Europe. It has a tendency to raise the strength and cause brittleness. It is considered undesirable when more than 0.1 per cent is present.

**693. Non-metallic Impurities.**—Steel and iron also frequently contain very small percentages of the oxides and silicates of both iron and manganese. These impurities are mechanically suspended in the metal and are often called slag inclusions. They appear to arise from trapping of the slag, from the cleansing action of the recarburizer, and from the spalling of the ladle and furnace linings. In well-made steel such inclusions are slight and of small moment. If segregated they are likely to cause brittleness.

\* See data presented by J. S. Unger in *Engr. News*, Vol. 75, p. 363.

† *Proc. A. S. T. M.*, Vol. 17, Pt. 2, p. 8.

From time to time the opinion has been advanced that nitrogen and hydrogen are present in iron and steel, especially in Bessemer steel, and that they embrittle the metal. Quantitative evidence of these facts is, however, lacking.

#### EFFECTS OF HEAT TREATMENTS

**694. Effects of Heating above the Critical Range.**—On heating steel past the critical range of temperature, there is a structural change from an aggregate of ferrite and cementite to a homogeneous solid solution. This transition effectively destroys all pre-existing crystallization, not only as to type of aggregate, but of size as well.

Referring to the Roozeboom Diagram, Fig. 1, Ch. XXI, it will be noted that so long as a steel of carbon content 0 to 2 per cent is not heated above  $Ar_1$  ( $690^\circ\text{C.}$ ) there will be no change in the structural relations of the iron and the cementite.\* With rise of temperature above this critical point, however, formation of austenite will begin, and proceed to a degree dependent upon the carbon content of the steel and the temperature attained above the critical one. Solution of all of the pearlite is completed immediately the temperature exceeds  $Ac_1$ .

For hypoeutectoid (below 0.9 per cent C) or hypereutectoid (above 0.9 per cent C) steels the solution of the excess ferrite or cementite, respectively, proceeds with each degree of temperature rise above  $Ac_1$ , but is not completed until the temperature reaches the upper transition limit, marked by  $GP$  and  $PS$ , respectively, corresponding to the particular carbon content of the steel under treatment. In other words, the austenitic state in a steel of eutectoid composition (0.9 per cent C) is entirely brought about by heating to or just above the constant critical temperature of transition; while for a steel of other carbon content, a range of temperature is necessary, which is greater the farther the carbon content is removed from 0.9 per cent. The temperature necessary to accomplish complete solution of the cementite is higher the more the steel varies either way from eutectoid proportions. The transition, as noted heretofore, is the result of allotropy in the iron; cementite is immiscible in alpha iron, but is completely soluble in gamma iron, within the saturation limits, marked by region IV in the Roozeboom Diagram.

**695. Effects of Cooling from above the Critical Range.**—There will be progressive aggregation of the austenite into grains which increase in size with rise of temperature, up to the fusion stage. That grain size which is attained as a result of the maximum temperature reached

\* On account of lag in reaction the change takes place at  $Ac_1$  which is slightly higher than  $Ar_1$ .

during heating, will be retained during the cooling of the steel to ordinary temperatures, and, although the transition from the austenitic to the pearlitic (or some intermediate) form will take place during the critical temperature range, the final structure will exhibit such coarseness of texture as was inherent in the austenite when cooling began. Coarseness of grain is a function of temperature rise above the critical point  $Ar_3$ ; but diminution of grain is not an accompaniment of temperature fall.

Rate of cooling through and somewhat below the transition zone influences the final structure, however; (1) the reversion from austenite to ferrite-cementite requires time for its completion, and the quicker the cooling for a given composition, the less complete is the transition, and the more nearly the final product approaches austenite in structure and properties; this will be treated more fully in the discussion of tempering of steel; (2) the ferrite-cementite aggregate, particularly the pearlitic portion, will have opportunity for greater coalescence of like to like, the slower the cooling.

**696. Relation of Grain Size to Mechanical Properties.**—It is considered almost an axiom that, other things being equal, strength and ductility are inversely proportional to the size of grain. The grains are aggregates of crystals, each of the same form and of like *orientation* which latter is not the same, except by coincidence, in the adjoining grains. The larger the separate aggregates making up the individual grains, the less there are for a given cross-section of material, and the more direct will be the path of fracture along the cleavage planes within the grain, or following the boundaries of contact of the separate grains. The weakness due to coarse grain is likely to be especially pronounced under vibratory or repeated stresses, since a fracture, once started, has a less broken course in attaining such dimensions that failure becomes inevitable even under loads which are apparently within the margin of safety. Coarse grain also lessens the resistance of steel to impact.

**697. Annealing.**—Practically all steel is actually cast into a mold as a liquid mass, and cools through the solidification, austenitic and pearlitic temperature ranges at rates which vary with the bulk of the section and the mold conditions. At best, the metal of necessity has cooled from a temperature of maximum intensity, which will tend to promote coarseness of crystallization, and consequent loss of strength and ductility. Slow cooling, particularly near the solidification zone, will aggravate the effect, because of the increased time under favorable conditions for growth of the austenitic aggregate. Untreated cast steel is inherently relatively weak, and in large measure this weakness is a function of the size of the casting.

The structure and properties may be improved by judicious annealing, by taking advantage of the obliteration of previously existing structure on heating just above the critical range ( $690\text{--}900^{\circ}\text{C.}$ ). The operation consists of heating the steel to the minimum temperature and for the minimum time needed to insure complete change of grain to the desired fine texture; then cooling it from this temperature at such a rate as will best conserve the wished-for structure and properties. The rate of heating should be slow enough to permit uniform diffusion of heat throughout the piece. The specific treatment

will vary with the amount of carbon and other constituents in the steel, with the size of the object, and with the relations of hardness, strength, and ductility, which are desired in the final product. Soft steels require a higher temperature than those more nearly approaching eutectoid proportions, since the complete conversion to austenite is affected only by heating past the upper critical temperature. Simple annealing of mild steels does not produce as great a refining of grains as is

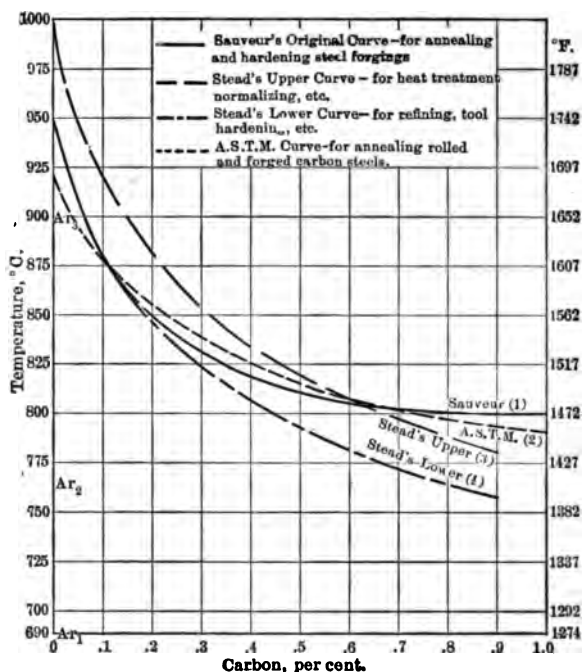


FIG. 10.—Temperatures for Annealing and Hardening Carbon Steels.

obtained with eutectoid steels, since the aggregation of the eutectic begins at the common lower critical temperature, while the higher end-temperatures for the lower-carbon steels allow greater coarseness of the newly-formed grain. The range of annealing temperatures suitable to various carbon steels is well shown by curves 1, 2 and 3 of Fig. 10.\* These diagrams show that authorities agree pretty closely on proper annealing temperatures for medium and high-carbon steels but there is

\* Modified from a figure by Stead (*Jour. Ir. and St. Inst.*, Vol. 94, p. 50) by the addition of the curve representing the annealing temperatures recommended by the A. S. T. M. for forged and rolled carbon steels.



a wider diversity of opinion concerning the most beneficial temperature for annealing low-carbon steels.

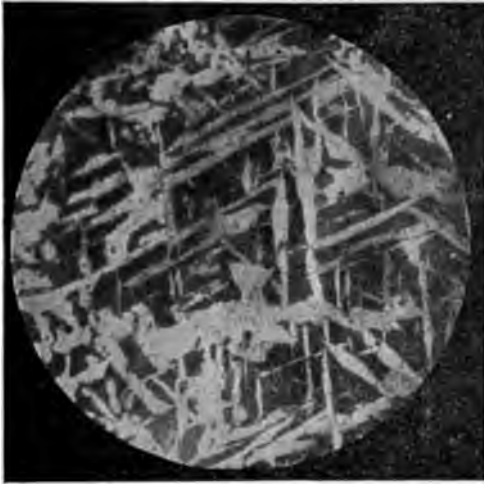


FIG. 11.—Coarse Structure in a Steel Casting  
Due to Slow Cooling.

fixes the grain size of the product. It should always be the lowest possible to effect complete refining. Rate of cooling from this temperature also has a pronounced effect on the physical properties. Rapid cooling results in incomplete reversion of the austenite to pearlite to a degree which varies with the carbon content of the steel and the rapidity of cooling. This is a process of hardening, and is treated in greater detail later. Provided cooling is slow enough to enable the steel to revert to the pearlitic condition, the aggregation of its cementite ferrite constituents will be coarser the slower the cooling, and the greater softness and ductility are thus obtained at a sacrifice of some strength and elasticity. Cooling in the furnace or imbedded in lime, clay, etc., is slow; quiet air or an air

For steels with a manganese content greater than 0.75 per cent, slightly lower temperatures than indicated by the A. S. T. M. curve suffice. Heating should be by gradual approach to the desired temperature, and the object should be held at this temperature for a sufficient time to reach a uniform condition throughout. This time varies with the size of object, about an hour for 12 inches of thickness should suffice.

Temperature of annealing

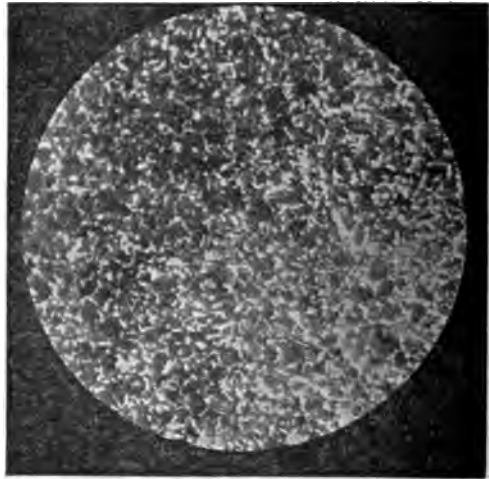


FIG. 12.—Same Steel Casting after Forging  
Showing a Much Finer Grain than Fig. 11.

blast is more rapid, and is usual for large objects unless abnormal softness is desired, or the carbon content of the steel is high.

Small pieces should always be protected from oxidation and consequently decarburization of the surface during annealing. This may be partially accomplished by surrounding the parts with lime, sand, or pulverized charcoal. It may be very effectively done by the Jones method. This consists in placing the material in a closed tube from which the air is expelled by a non-oxidizing gas which is kept constantly flowing through the tube. Surfaces of parts thus treated are bright and untarnished. Metcalf also uses a tube closed with a loosely-



FIG. 13.—Same Steel Casting Annealed at too High Temperature ( $1100^{\circ}\text{C.}$ ). Note the Coarse Grain.

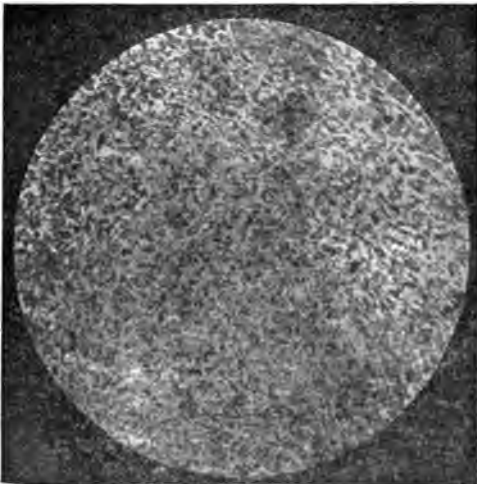


FIG. 14.—Steel Casting of Fig. 13 after Annealing at  $800^{\circ}\text{C.}$

again produced a coarse grain (Fig. 13). A fine grain (Fig. 14) was given this overheated steel by annealing at  $800^{\circ}\text{C.}$ , which is approx-

fitting cap. The air is almost entirely expelled from the tube by the volatilization of resin which is placed in the end of the tube farthest from the cap. Parts annealed by the Metcalf method are slightly tarnished but decarburization is negligible.

The changes in size of grain brought about by certain heat treatments of a piece of 0.50 carbon steel are shown in Figs. 11 to 16. The coarse structure of the steel as cast (Fig. 11) was materially reduced by forging (Fig. 12), but reheating of the forged steel to  $1100^{\circ}\text{C.}$

imately the proper temperature for annealing this grade of steel. (See

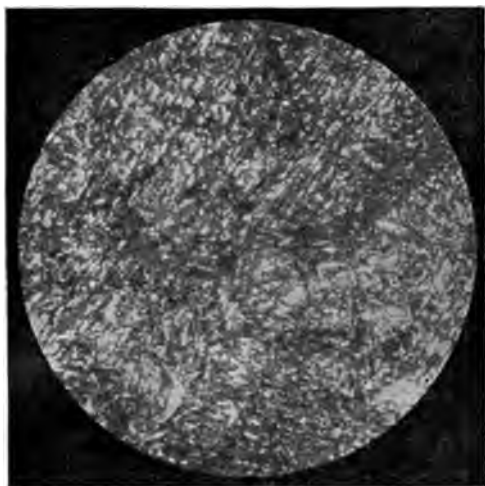


FIG. 15.—Same Steel Casting Heated to 1000° C. and Quenched in Cold Water Shows a Martensitic Structure. Structure indicates high strength, hardness, and brittleness.

Fig. 10.) A still finer grain showing a very tough steel was produced by a double quenching treatment. The steel was first quenched in water from a temperature of 1000° C. which made it very hard and gave it a fine structure appearing like interlacing needles and called martensite (Fig. 15). Subsequently it was heated to 650° C. and again quenched in water, with the result shown in Fig. 16. This represents the fine porcelainic structure of sorbite in which the transition from martensite to pearlite plus ferrite has been practically completed

but without opportunity for coalescence of pearlite masses and ferrite grains as in annealing at higher temperatures. Sorbite is the structure which is characteristic of the toughest steels.

#### 698. Effects of Annealing on Mechanical Properties.—

Referring again to Figs. 4 and 5 it will be observed that annealing from temperatures specified by the A. S. T. M. (See Fig. 10) reduces the strength, hardness, and elastic ratio but increases the ductility. The effects are most pronounced in steels having more than 0.5 per cent carbon and are of small moment in the very low-carbon steels.

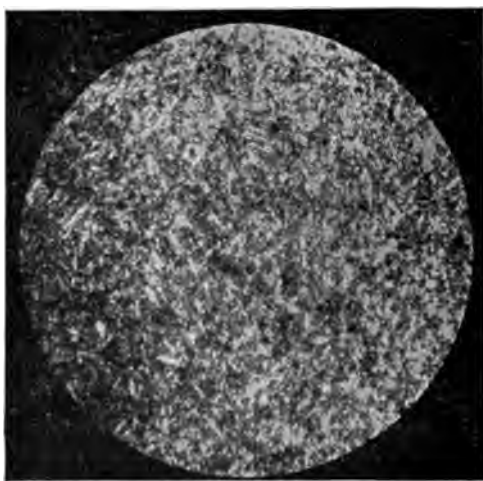


FIG. 16.—After Reheating the Casting of Fig. 15 to 650° C. and Quenching, a Sorbitic Structure Results. This structure connotes high strength and toughness.

The effect of temperature of annealing on the mechanical property

of several tool steels is well illustrated by the experiments of S. v. Fabry, from which Fig. 17 has been prepared. Here again it will be noted that the properties of the high-carbon steels are much more affected by annealing than those of the medium-carbon steel. For all cases, it will be noted that the ductility is a maximum for temperatures at which the strength is a minimum and that the elevation in the per cent reduction in area is also most prominent at the same temperatures. These temperatures range from 700° for the 0.58 per cent carbon steel to 800° C. for the steel of 1.36 per cent carbon. It should also be observed that the minimum strengths of these steels are nearly the same but the corresponding percentages of reduction in area diminish as the carbon content increases. In general, the

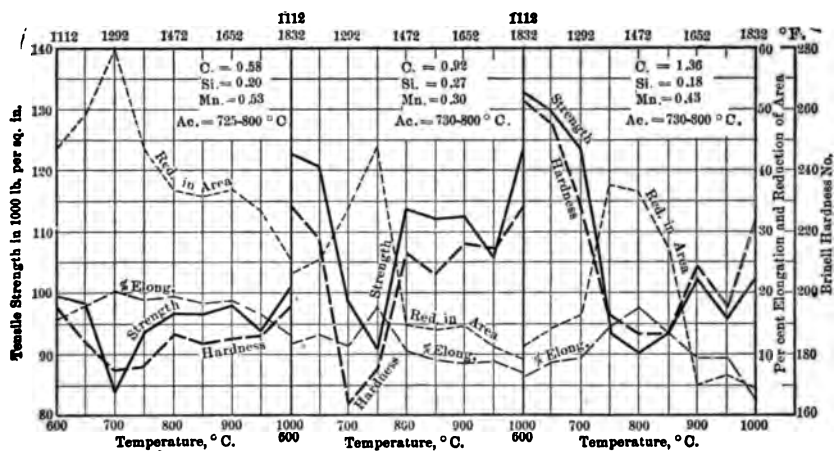


FIG. 17.—The Influence of Annealing Temperature on the Mechanical Properties of Tool Steels. (Fabry in *Proc. I. A. T. M.*, 1912.) Specimens 1.18 in. square were held at temperatures indicated for three hours.

curves of hardness and strength are nearly parallel. The microscopic data accompanying the original report indicate that the most uniform and the finest grained structures were produced by annealing at the temperatures corresponding to maximum ductility or minimum strength.

The effects of variations in annealing temperature on the strength and ductility of steels differing in carbon content are also shown in Fig. 29 and 30. From these data by Brinell it appears that the strength of the low and medium-carbon steels was little affected by any of the annealing temperatures. The strengths of the high-carbon steels were slightly increased by annealing at 350° C., slightly diminished by annealing at 750° C., and considerably increased by annealing at 1000° C. The ductility of all grades of steel was raised somewhat by annealing at 350° C. Further increase in ductility was secured

by annealing at 750° C. Annealing at 850° C. produced the most marked increase in the ductility of the high-carbon steels. Annealing at



FIG. 18.—Showing Coarse Grain Produced in Rail of Fig. 19 by Overheating.

higher temperatures effected a corresponding slight increase in the ductility of low-carbon steels, but the curves plainly show that these temperatures were too high to secure maximum ductility from the high-carbon steels.

**699. Overheating and Burning.**—Exposing steel to high temperatures for long periods develops a very coarse grain; but so long as the temperature does not enter region II of the Roozeboom Diagram (Fig. 1, Ch. XXI) for the particular carbon content, it is possible to

effect complete refining by reheating above the critical range. The structure of Fig. 18 could be restored to that of Fig. 19 by proper annealing. If, however, the temperature of heating is such as to take the steel into region II, partial fusion results, and the steel becomes "burnt." There is mechanical separation of the grains due to the partial fusion and gas evolution, with probable accompaniment of some oxidation of the boundaries. Thus the steel becomes brittle or "rotten." Such an effect is not curable by reheating, or even by reheating and forging. The differences between burnt, overheated, and normal grains of a steel casting are well shown in Fig. 20.

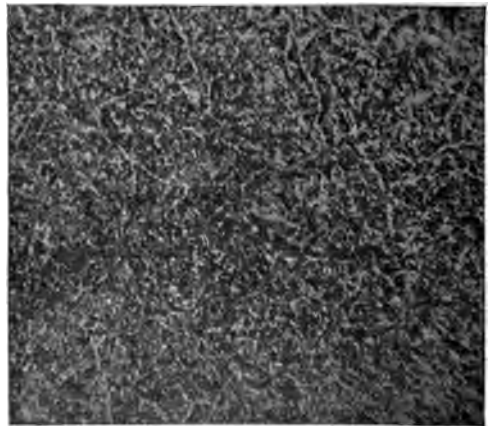


FIG. 19.—Structure of Rail after Rolling.

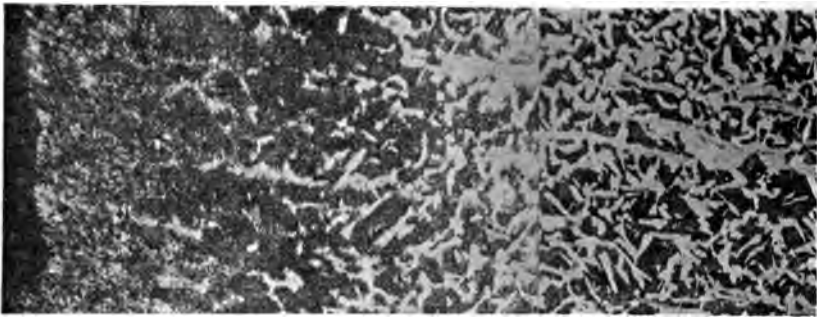
Although microscopic examination of sections of the metal subjected to overheating or burning will reveal the defective structure, it is not always convenient to make such an examination. The Metcalf test is simpler and effective in detecting overheating or burning in steels containing over 0.30 per cent carbon. For this test it is preferable

to use a bar of steel about a foot long having the same carbon content as the mal-treated piece, although the piece in question may be used. On opposite faces of the bar saw slots are cut to a depth of approximately one-eighth of the thickness of the piece at intervals of about one inch. One end of the piece is then struck in a fire and heated until it scintillates. Upon removal it may be quenched or allowed to cool in air, depending upon treatment accorded the piece in question. The test-piece is then broken and, beginning at the burnt end, the fractures at the notches show a gradual change from very coarse grain to a fine silky structure for the section which was at the proper restoration temperature (i.e., just above the critical range); sections nearer the cooler end will show the original structure of the bar. A comparison of the fracture of the piece in question with the series of fractures indicates the treatment accorded.

**700. Theories of Hardening.**—It has been known for centuries that some kinds of iron become very hard after heating to a bright red heat

Burnt. Overheated.

Normal Structure.



Actual width = 0.08 in.

FIG. 20.—A Steel Casting of 0.40 Per Cent Carbon Cut by an Oxyhydric Torch. Note gradation in structure from burnt metal at cut (on the left) to normal structure at right end.

and quenching in water. In fact, this property was the basis of the older classification of certain products as "steel." Also, it has been recognized for a long time that this property is a function of the carbon content of the steel, and that no appreciable hardening accompanied the quenching of iron with negligible or very small amounts of carbon. But it is only since the development of our understanding of the iron-carbon relations that the hardening of steel by quenching has been put upon our present-day fairly rational basis, and that the heat treatment of steel has deviated from the method of "rule of thumb."

Heat-treatment methods have reached a high degree of perfection; and the phenomenon as a whole is fairly well understood. Several theories of hardening are to-day in existence, each having its adherents and points of merit; yet none offer satisfactory explanation of all of the observed facts. Sauveur\* classifies them as follows:

\* *Metallography and Heat Treatment of Iron and Steel*, p. 308.

Retention Theories	{	Solution Theories	{	Beta Iron or Allotropic Theory.
				Alpha Iron Theory.
	{	Amorphous Iron Theory.		
Stress Theories	{	Carbon Theories	{	Hardening Carbon Theory.
				Sub-carbide Theory.
	{	Early Stress Theory.		
	{	Interstrain Theory.		
	{	Twinning and Amorphous Iron Theory.		

The retention theories are based upon the restraining of certain reactions which take place normally when steel cools through the critical temperature ranges. Very quick cooling through the transformation ranges suppress the change, and the condition stable only at higher temperatures is retained to a greater or less degree in the cold rigid steel. According to the several theories, this may be (a) a hard, beta allotropic form of iron normal only to a limited temperature range between 780° and 880° C., which is assisted by solution of iron carbide  $\text{Fe}_3\text{C}$ , and its influence in restraining the normal transition; (b) a metastable solution of  $\text{Fe}_3\text{C}$  in alpha iron, suppressing, therefore, the separation of the carbide, but not the change from gamma to alpha iron; (c) a hard amorphous transition form, which is intermediate to the complete allotropic change in the iron, and which is aided in its effect by the solution of the combined carbon and its influence in slowing down the reaction velocity; (d) an allotropic form of iron carbide, inherently hard, or conferring hardness because of its forced solution in the iron; (e) a hard sub-carbide of iron  $\text{Fe}_4\text{C}$ , stable only above the critical range.

According to the stress theories, steel cooled quickly from above the critical range is subjected to stresses due to the shrinkage of its outer shell on the interior, and to the increased volume accompanying the transformation of gamma into beta and alpha iron. Hardening accompanies the resultant straining of the metal. Theories in this class were among the first to be put forward to explain the hardening of steel and have recently attracted renewed attention. Hypotheses are advanced that (a) the transition of combined carbon from solution in gamma iron to non-solution in alpha iron is not completed, non-homogeneity of crystalline orientation is caused, and a hardness similar to that caused by interstrain is the result; (b) that severe internal strains due to quenching steel, cause crystal twinning and hard amorphous layers.

Many of the theories have much in common, and in some cases the distinctions are in large part technical or a matter of definition. The technical points are exceedingly complex and involve the highest principles of metallography in experimentation and elucidation. To quote Sauveur (*Metal. and Hard. of Steel—Int. Eng. Cong.*, 1915). "It will be obvious from the foregoing that the many attempts at arriving at a satisfactory explanation of the hardening of steel are based on one or more of the following conceptions; (1) existence of a hard allotropic variety of iron, (2) existence of solid solutions involving the occurrence of so-called 'hardening' carbon, and (3) existence of strains in quenched steel causing or not an amorphous condition of the iron."

"It will likewise be obvious that nothing so far presented fully satisfies our craving for a scientifically acceptable explanation of the many phenomena involved."

"It would seem as if the methods used to date for the elucidation of this complex problem have yielded all they are capable of yielding and that further straining of these methods will only serve to confuse the issue, a point having been reached when this juggling, no matter how skillfully done, with allotropic, solid solution, and strains, is causing weariness without advancing the solution of the problem. The tendency of late has been to abandon the safer road of experimental facts and to enter the maze of excessive speculations, in which there is great danger of becoming hopelessly lost. The

conclusion seems warranted that new avenues of approach must be found if we are to obtain a correct answer of this apparent enigma."

Many of the theories are founded upon special or unusual conditions; others, again, fit the general observations but fall short of covering all of the facts. It seems logical to assume that since hardening in metals (including steel) can be the result of several physical effects, observed phenomena are not to be explained by theories based upon one principle only; there may be several contributing causes.

**701. Essentials in Hardening.**—The operation of hardening of steel consists of two essential steps: (1) Heating above the critical range of temperature; (2) cooling rapidly from that temperature. No change in the iron-carbon relations will take place until the steel is heated to the lower critical temperature  $A_{c1}$  (about  $700^{\circ}\text{C.}$ ). Austenite will then begin to form, and the reactions will be completed at the upper transition temperature  $G_{PS}$  of the Roozeboom Diagram, Fig. 1, Ch. XXI. As indicated by the diagram, and discussed more fully Art. 661 complete conversion of a eutectoid steel (0.9 per cent C.) may be effected by heating to just above the constant temperature  $A_{c1}$ ; while for steels of other carbon content, a range of temperature is necessary, which becomes greater as the carbon content is farther removed either way from eutectoid proportions (Fig. 10).

The above transitions are reversible, under normal conditions, and the reactions will reach a condition of equilibrium with rise or fall of temperature, in accord with the constitution diagram. The essential reaction is ferrite plus cementite to austenite, or the reverse; or combined carbon ( $\text{Fe}_3\text{C}$ ) into or out of solution with gamma iron. The reaction requires time; separation is relatively much slower than solution, and naturally requires longer time in proportion to the quantity of cementite to separate; that is, with increase of carbon content in the steel. Again, the reaction velocity of separation becomes less as the steel becomes colder and more rigid. Therefore, by suddenly cooling a steel which is in the austenitic temperature range, it is possible to bring it to such a temperature and state of rigidity that the reaction velocity becomes nil, and the austenite will be forcibly restrained in the steel at normal temperatures, in its entirety or in such intermediate transition form as might result from the carbon content and cooling conditions.

**702. Methods of Hardening.**—The quenching capacity of a medium for hardening steel depends upon its specific heat, conductivity, volatility, viscosity, and temperature. The hardening bath should always be kept at a uniform temperature, especially if water is used, and should be continually circulated to prevent the formation of vapor envelopes about the metal, since these materially retard the withdrawal of heat. In order of hardening capacity the more common quenching media rank thus: water spray, brine, water, oil and molten lead. The



first two of these are used commercially on small parts only, when an extremely hard surface is desired. Files are often hardened in brine. Water quenching likewise is so drastic that its use is restricted to the low-carbon steels or to very small parts of simple shape made of high-carbon steel. With the heavy sections of high-carbon steel these drastic hardening agencies are likely to cause cracking.

On account of the wide range in quenching power which various oils possess and because their action on the steel is less severe than water quenching, they are much used for hardening tools, machine and structural parts where toughness is an important consideration. In some works the drastic cooling of the water-bath is somewhat

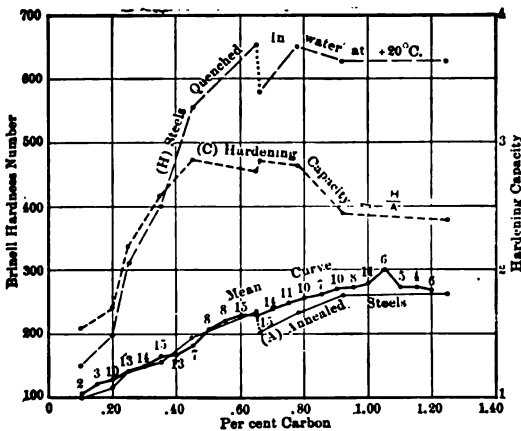


FIG. 21.—The Relations between Carbon Content, Hardness and Hardening Capacity,  $\frac{H}{A}$ . (Brinell in *Jour. Ir. and St. Inst.*, Vol. 59, p. 243.)

Figures beside plotted points indicate number of results averaged. The silicon content varied from 0.1 to 0.5 and manganese from 0.2 to 0.6 per cent in the steels represented by the mean curve. The discontinuity in curves H, C, and A at 0.66 per cent carbon is due to the lower manganese contents in the higher carbon steels.

hardening. This is often advantageously done in heavy shafts and axles. The inferior metal at the center of the piece is thus removed; and, by forcing a stream of oil through the hole during quenching, a much more uniform hardening treatment is gotten.

**703. Effect of Carbon on Hardening.**—The influence of carbon in hardening steel is well brought out by the experiments of Brinell, some of the results of which are presented in Fig. 21.\* It will be observed that the hardening capacity is greatest and nearly constant for steels

relieved by floating an oil bath above it; in others the parts are partially cooled in water before being quenched in oil. The principal use of molten lead is for toughening the metal.

Parts of irregular form are likely to warp and crack when rapidly hardened. Long pieces should be immersed in the bath end first wherever possible. If very long, such pieces may be allowed to roll into the bath. Inasmuch as it is almost impossible to harden uniformly pieces of large section they should, wherever possible, be bored before

\* See also Figs. 4 and 5.

containing 0.40 to 0.80 per cent carbon, but the maximum hardness is found in hardened steels of approximately eutectoid composition.

**704. Characteristic Microscopic Structures in Hardened Steels.**—Austenite, a solid solution of cementite in gamma iron, is hard; that is, it is harder than the ferrite, but not so hard as the cementite, which is the hardest of the constituents usually found in steel. Again, the hardness of the austenite increases with the amount of cementite in solution. In reverting to the pearlitic conditions normal at ordinary temperatures, the separation of ferrite and cementite from solution,



FIG. 22.—A 1.5 Per Cent Carbon Steel Quenched in Brine from 1200° C. Mag. = 150 d. Showing troostite in black rounded areas with surrounding dark needle-like particles of martensite set in a white background of austenite. (Courtesy of Sauvour and Boylston, see Fig. 286 in *Metal. and Heat Treatment of Iron and Steel.*)

with subsequent re-arrangement into the final state of aggregation, is a process of gradual progression, and requires for its completion an interval of time, or of temperature, or both. While these transitory stages of dissolution and of coalescence are not to be considered as new constituents, and there are no well-defined limits of division, convenience in description has led to the introduction of the names, martensite, troostite, and sorbite for certain fairly characteristic structures in the re-arrangement. The transition found in hardened steels have the following distinguishing features:

*Austenite* is a solid solution of  $\text{Fe}_3\text{C}$  in gamma iron. It may also include other ele-

ments, manganese, nickel, etc., in association in the solution. The carbon may vary from traces to 1.75 or 2.0 per cent. Austenite is a normal constituent of all carbon steels above the critical temperature range, and may be retained in varying proportions by rapid cooling by quenching from these high temperatures. These proportions depend upon the amount of carbon, temperature of quenching, and speed of cooling. In plain low-carbon steels, there is no retention of austenite, due to the rapidity of transformation; with high carbon (above 1 per cent) a proportion up to about 50 per

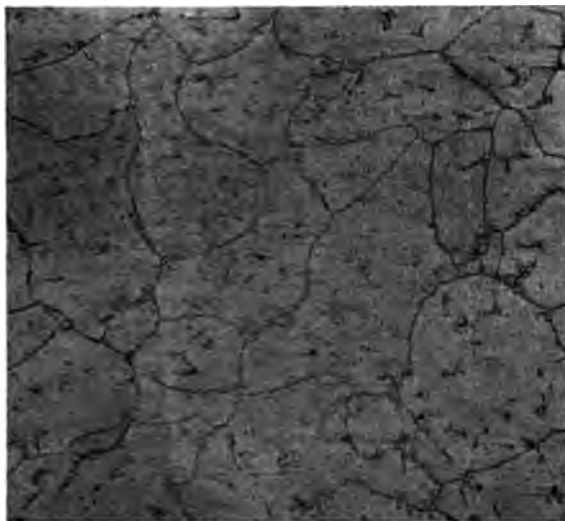


FIG. 23.—Cast Manganese Steel Quenched in Water from 1000° C. Nearly 100 per cent austenite. Mag. = 100 d. (From Fig. 325, Sauveur's *Metal. and Heat Treatment of Iron and Steel*.)

cent may be checked in association with martensite, the first transition product. The structure of this type is given in Fig. 22.

Associated metals, as manganese, nickel, tungsten, etc., in sufficient amount lower the critical temperature or slow down the rate of transition, so that it will not take place at all or will be checked by moderate cooling, with retention of austenite alone. Being a homogeneous solid solution it exhibits the characteristic polygonal grain structure, Fig. 23, is a typical microsection.

The physical properties of austenite vary with the carbon content; an increase of this element

increases the hardness, strength and elasticity and lowers the ductility. Austenite is hard relatively to ferrite, but is softer than cementite, or than martensite of the same carbon content. Austenitic steels are non-magnetic, tenacious, rather ductile, and tough.

*Martensite* is the first stage in the decomposition of austenite. It is the principal constituent of hardened carbon steels, and the cause of their great hardness. There is conflict with regard to its constitution; allotropists contend that it is a solution of carbon in beta iron, with some reversion to alpha iron, the latter accounting for its magnetic susceptibility. The greater hardness of martensite compared with austenite is the bulwark of the defense of a hard, allotropic form of iron (beta) and its influence on the hardening of steel. On the other hand there is strong support of the argument that beta iron does not exist, and that the hardness of martensite is the natural accompaniment of the solid solution state, augmented in turn by the metastable condition resulting from complete or partial reversion of the gamma to alpha iron, wherein the latter cannot assume its molecular symmetry and orientation, thus bringing about interstrain and correlative hardness.

Martensite is obtained, associated with austenite, by quenching very high-carbon steels from high temperature in ice water, or in total by water-quenching eutectoid or hypereutectoid steel from the austenitic temperature range. Microscopically, as shown

in Fig. 22, the structure appears as a network of interlacing needles of triangular distribution, assumed to be cleavages of octahedra. Martensite is very hard, very strong, brittle, and non-malleable when cold. These properties are intensified with increasing additions of carbon.

*Troostite* is the second transition stage, associated with martensite as a result of partial tempering after hardening or of quenching medium carbon steels in the austenitic region, or high-carbon steels within the transition range. It appears as nodular aggregates bordering the martensite grains (see Fig. 22), or associated with sorbite, and is darkly colored by etching.

The constitution of troostite is a matter of controversy, but the physical and chemical properties, between those of martensite and sorbite, indicate that it is a mixture of cementite and ferrite, differing from pearlite only in the state of division.

*Sorbite* is the last stage in the transition from austenite to cementite-ferrite, and considered to be pearlite in a minutely granular form. It etches to a uniform dark-colored mass, with finely granular structure at high magnifications, as indicated in Fig. 24.

Sorbite forms in lighter sections of steel cooling in air from above the critical range, by oil quenching pieces of medium carbon, or by water quenching from the lower parts of the transition zone, or by reheating hardened steel almost to the critical temperature.

Because of their fine and homogeneous state of aggregation, sorbitic steels have high strength and elasticity coupled with maximum ductility compared with those of normal pearlitic structure.

*The fundamental reaction* which takes place when a quenched steel is restored to equilibrium is solid solution to separation of cementite and ferrite. This occurs in progressive steps thus:

Austenite  $\rightarrow$  martensite  $\rightarrow$  troostite  $\rightarrow$  sorbite  $\rightarrow$  cementite + ferrite.

Naturally, with no sharp boundaries between the constituents, tempered steel may be relatively complex, and consist of various associations of the transition forms. The extreme hardness and brittleness of martensite steels make them practically unsuited for industrial uses, except cutting tools; even here, and to a much greater extent in structural products, varying proportions of the other transition forms are desired. They are obtained by proper selection of composition of steel, maximum temperature of heating, nature of cooling medium, and by tempering; that

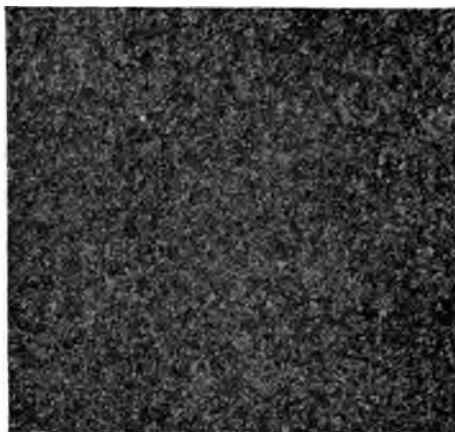


FIG. 24.—A 1.1 Per Cent Carbon Steel Quenched from 900° C. Reheated to 600° C. and Quenched, Showing Nearly 100 Per Cent Sorbite. Mag. = 150 d. (Courtesy Sauveur and Boylston.)

close as practicable to the critical range. The latter varies somewhat with different steels, and is best determined from cooling-curve data obtained with a pyrometer. For steels of moderate or high-carbon content (above 0.50 C.) loss of magnetism in the heated steel coincides with the critical temperature; thus a magnet serves as a detector, since it will not be attracted by the heated object if the transition zone has been passed.

The drawing temperature is most accurately indicated by a pyrometer; but in tool dressing, color methods give a simple and reasonably accurate control. If a piece of hardened cold steel is brightened and

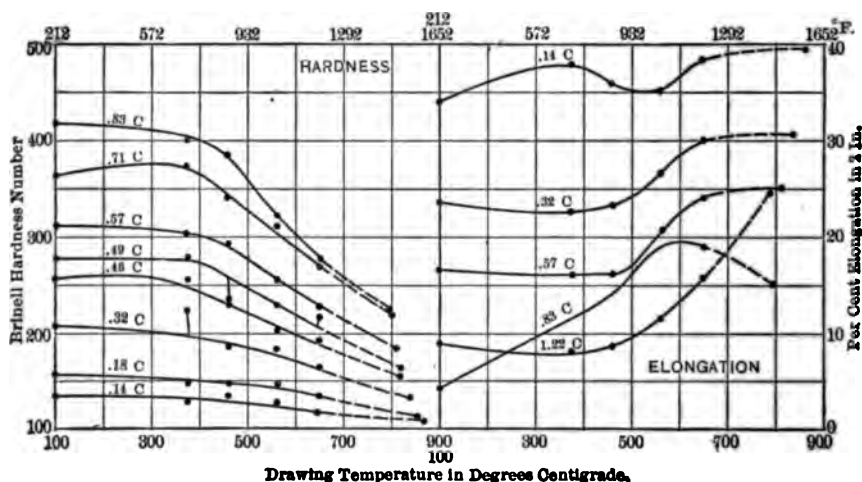


FIG. 28.—The Effect of Drawing Temperature on the Hardness and Elongation of Oil-quenched Steels. (Nead.)

placed in a hot furnace, the surface will assume oxide tints varying through pale straw color, brown and blue to the final blackish scale color, as the temperature of the steel rises; and it will become progressively softer. The approximate relations of temperature and color, both oxide and incandescence, are noted in the accompanying table.

Color of Oxide.	Deg.C.	Deg.F.	Degree of Incandescence.	Deg.C.	Deg.F.
Light straw.....	221	430	Red—visible in dark.....	400	752
Straw.....	232	450	Red—visible in twilight.....	474	885
Dark straw.....	243	470	Red—visible in daylight.....	524	975
Yellow brown.....	254	490	Red—visible in sunlight.....	581	1078
Dark brown.....	266	510	Dull cherry red.....	800	1472
Brown purple.....	271	520	Cherry red.....	900	1652
Dark purple.....	277	530	Bright red.....	1000	1832
Bright blue.....	288	550	Orange yellow.....	1200	2192
Full blue.....	293	560	Yellow white.....	1300	2372
Dark blue.....	316	600	Welding white.....	1400	2552

Most tools of ordinary carbon steel are treated by heating to a temperature slightly above the critical range (Fig. 10); the edge of the tool is hardened, by quenching in water or oil, withdrawn from the bath and brightened. The surplus heat in the red-hot unquenched portion is allowed to reheat the hardened quenched part until the proper temper color has reached the edge, the whole tool is then quenched to check further drawing of temper. This method is best for edge tools, such as lathe tools, since it gives the proper hardness at the edge, backed by a softer and tougher portion better adapted to withstand shock. However, it is unsuited for taps, milling cutters, etc., which must have uniform or distributed hardness; in this case quenching of the entire tool and subsequent reheating in the furnace are necessary. To avoid cracking, large sections, such as axles or shafts, should never be quenched to room temperature if they are to be placed in a hot drawing furnace, but should be held at 100 to 150° C. For carbon steels, the initial heat for quenching is preferably carried to the lowest temperature at which hardening will take place, since there will be the accompaniment of minimum coarsening of grain, and resultant maximum toughness of product.

Reheating of the hardened steel parts may be done in a furnace on a hot plate, in a sand bath or in a liquid bath. For drawing temperatures under 300° C. oil is commonly used. For higher tempering temperatures a salt bath made of 2 parts potassium nitrate to 3 parts sodium nitrate or a lead bath is often employed.

Heating of the metal either in hardening or drawing should be at a sufficiently slow rate to secure a uniform temperature throughout the piece.

**708. Drawing Temperatures for Various Classes of Steels.**—The degree of tempering will vary with the nature of service, being greatest for tools subjected to shock, and where great hardness is less essential than toughness. Stoughton \* gives the following drawing temperatures for various classes of tools:

225° C. Faint Straw	}	Tools for metal planers, small turning steel, and wood engraving.
to to		
235° C. Dark Straw	}	Punches and dies, taps and dies, milling cutters, boring tools, reamers, wood machine tools rock drills.
236° C. Dark Straw		
to to		
250° C. Light Brown	}	Twist drills, wood tools.
251° C. Light Brown		
to to		
275° C. Purple	}	Cold chisels, wood chisels, axes, metal and wood saws.
276° C. Purple		
to to		
300° C. Blue		

\* *The Metallurgy of Iron and Steel*, p. 384.

critical zone, result in physical properties which can be varied within wide limits between those of the hardened steel and those of the normal pearlitic state.

*Brinell's Strength Tests.* The data furnished by Brinell's\* experiments provide some good illustrations of characteristic influences of various heat treatments on the mechanical properties of steels. Some of his data have been plotted in Fig. 2), 30, and 33. From Fig. 29 and 30 it follows that water quenching from above the critical zone without subsequent drawing causes an increase in the strength and a loss in the ductility of the low- and medium-carbon steels and for these steels is more effective than oil quenching. (It should be borne in mind, however, that such quenching when applied to larger pieces of steel is likely to produce heavy internal stress and may even cause cracking.) For the high-carbon steels the oil-quenching treatments gave greater strength and ductility than quenching in water at corresponding temperatures. Water quenching of the high-carbon steels caused warping and heavy internal stress. Under such conditions the specimens were eccentrically loaded in testing and, being very brittle, broke at comparatively low stresses. In Brinell's quenching and drawing treatments it appears that drawing at 250° C. after quenching in oil at 80° C. had about the same effect on strength and ductility as quenching in water at 20° C. and tempering at 550° C.

Quenching from 850° C. followed by tempering gave the best combination of strength and ductility of the treatments tried by Brinell. This is especially true for the water-quenched specimens. It is likely that quenching the high-carbon steels at a somewhat lower temperature and the low-carbon steels at a somewhat higher temperature could have given still better results. It will be observed that in the case of water-quenched specimens the drawing temperature necessary to give the greatest strength increases with the carbon content. For example, consider the plotted data in Fig. 29 for steels water-quenched from 850° C. The 0.09 per cent carbon steel was strongest when quenched, the 0.25 and 0.44 per cent carbon steels were strongest when drawn at 350° C., and the other steels had highest strength when drawn at 550° C. It is likely that high-carbon steels would have had still greater strength if drawn at 400 or 450° C.

*The influence of tempering on the strength and ductility of oil-quenched steels* is well brought out by the results of Nead's tests.† In these experiments quenching was done in accordance with the recommendations of the A. S. T. M. (see curve in Fig. 10) and the results are very uniform. In Fig. 31 it will be observed that the ultimate strength

\* *Jour. of Ir. and St. Inst.*, 1901, Pt. 2. p. 234.

† *Trans. Am. Inst. of Min. Eng.*, Vol. 53, p. 218.

and yield point are not materially affected by tempering at temperatures under  $400^{\circ}\text{C}$ . Fig. 28 shows that the ductility, excepting the

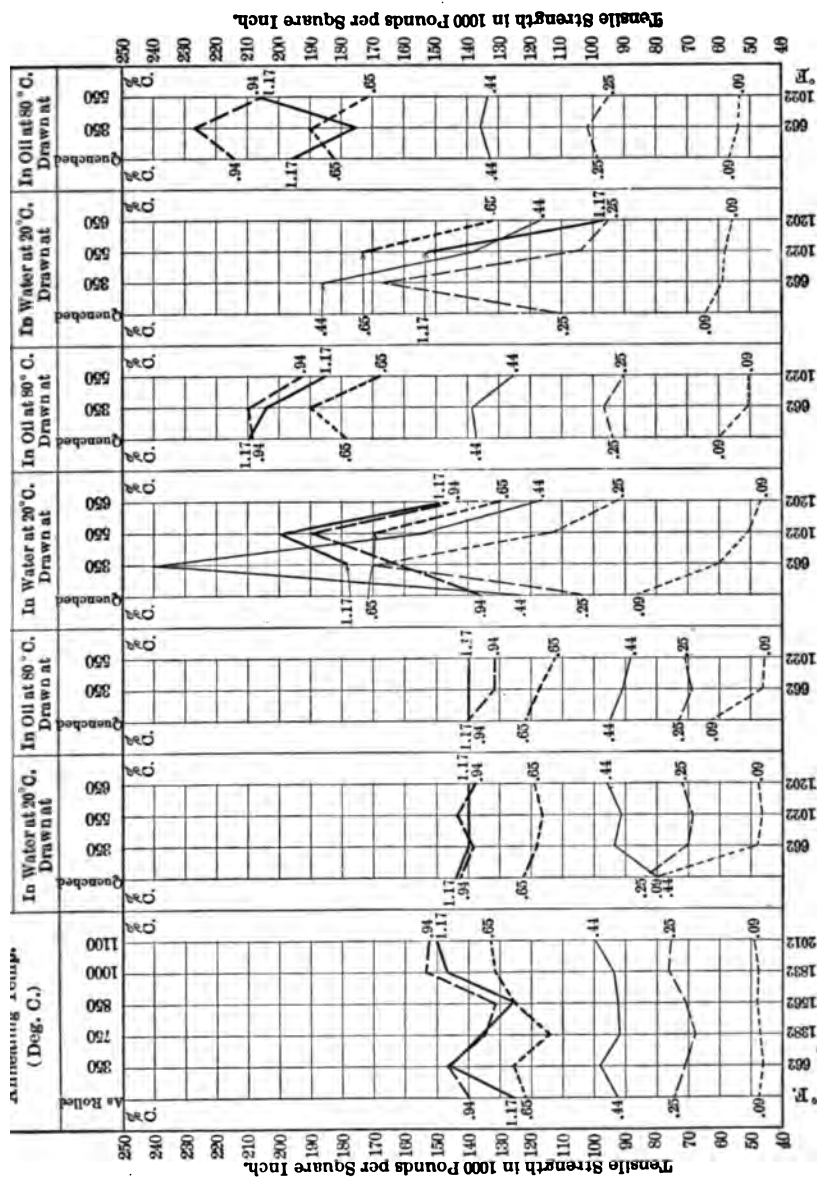


Fig. 29.—The Effects of Various Heat Treatments on the Tensile Strength of Acid Open-hearth Steels. (Brinell.)  
In most cases each point represents a single test. Specimens were 0.71 in. in diam. Composition limits were Si = 0.005–0.013, Mn = 0.10–0.48, S = 0.012–0.020, P = 0.0250–.029.

0.63 per cent carbon steel, is also unaffected by tempering within this range. There is produced, however, a marked decrease in strength and elastic limit and a corresponding increase in ductility as the drawing temperature is raised above  $400^{\circ}\text{C}$ . These effects are most pro-



critical zone, result in physical properties which can be varied within wide limits between those of the hardened steel and those of the normal pearlitic state.

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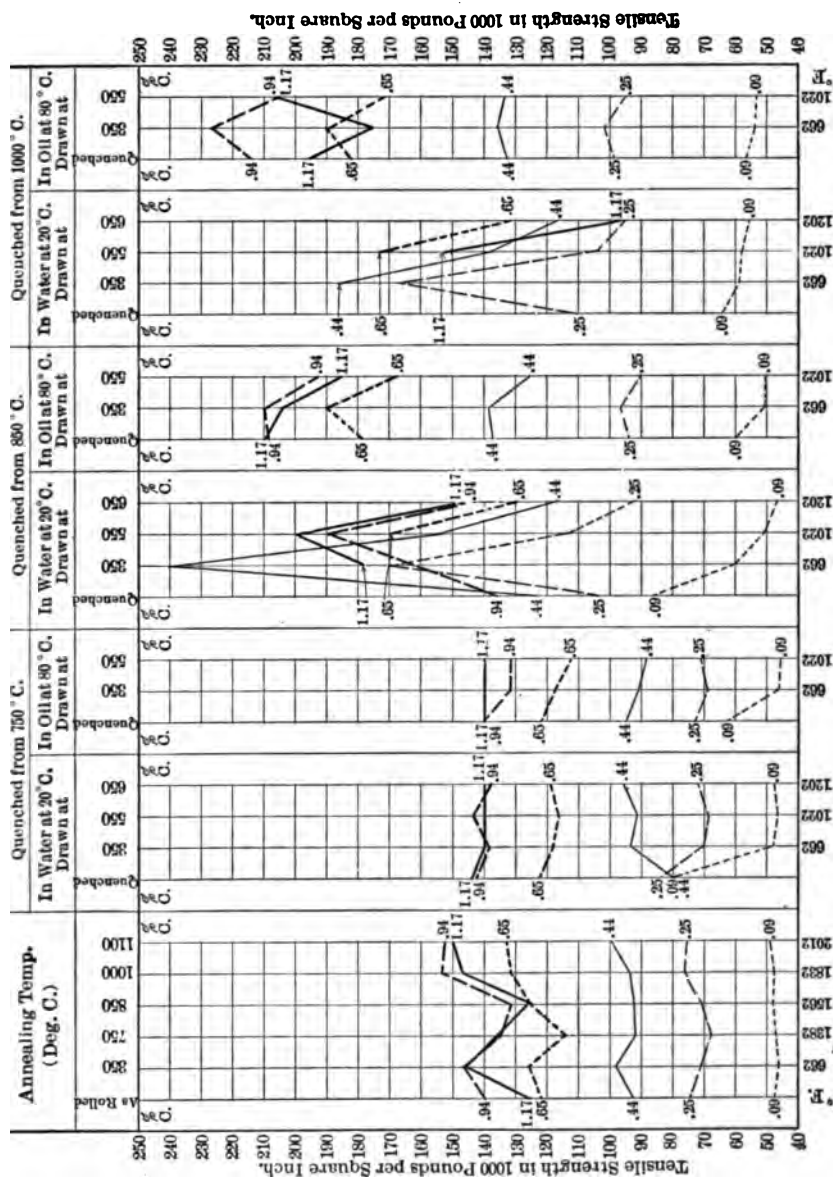


Fig. 29.—The Effects of Various Heat Treatments on the Tensile Strength of Acid Open-hearth Steels. (Brinell.)  
In most cases each point represents a single test. Specimens were 0.71 in. in diam. Composition limits were  $Mn = 0.005-0.313$ ,  $S = 0.012-0.020$ ,  $P = 0.0250-.029$ .

0.63 per cent carbon steel, is also unaffected by tempering within this range. There is produced, however, a marked decrease in strength and elastic limit and a corresponding increase in ductility as the drawing temperature is raised above 400° C. These effects are most pro-

nounced in the high-carbon steels, as would be expected. For larger pieces strength and ductility would be less affected by quenching, and constant for a greater range in drawing temperature.

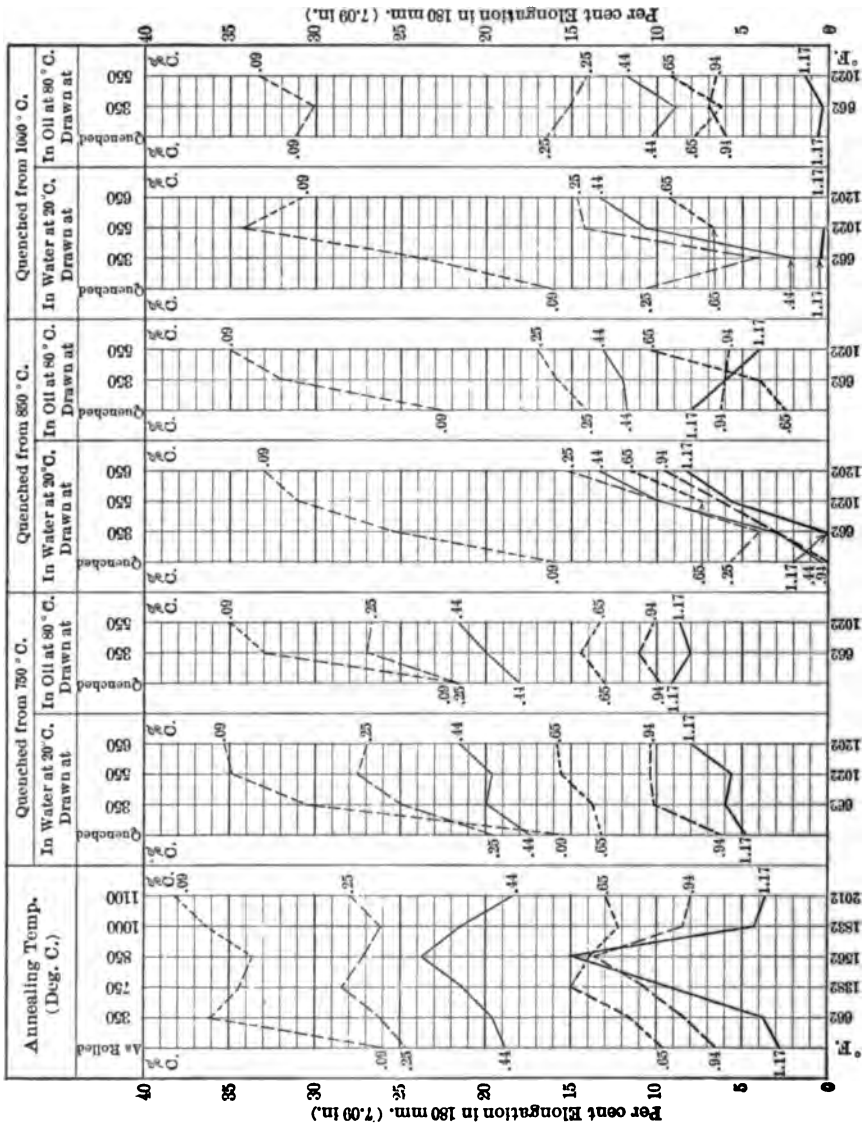


FIG. 30.—The Effects of Various Heat Treatments on the Elongation of Acid Open-hearth Steels. (Brinell.)

The toughening effect of quenching followed by drawing at various temperatures is admirably illustrated by the tests of Grard\* which are summarized in Fig. 32. The curves for energy of rupture in impact

\* Proc. I. A. T. M., 1912, III.

show that the low-carbon steels are the toughest and that maximum toughness in each of the steels tested was produced by drawing at a temperature about  $50^{\circ}\text{C}$ . below the upper critical point ( $A_{c3}$ ).

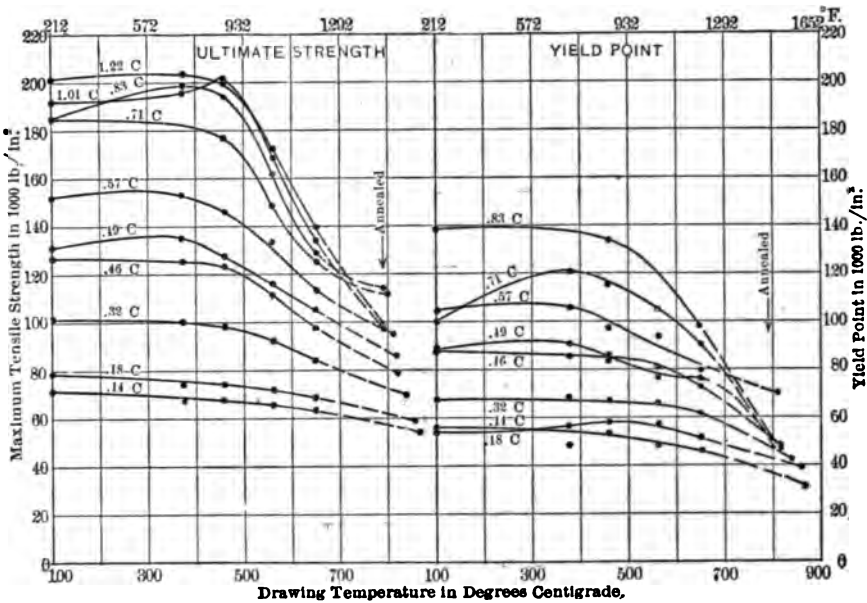


FIG. 31.—The Effect of Drawing Temperature on the Strength of Oil-quenched Steels. (Nead.)

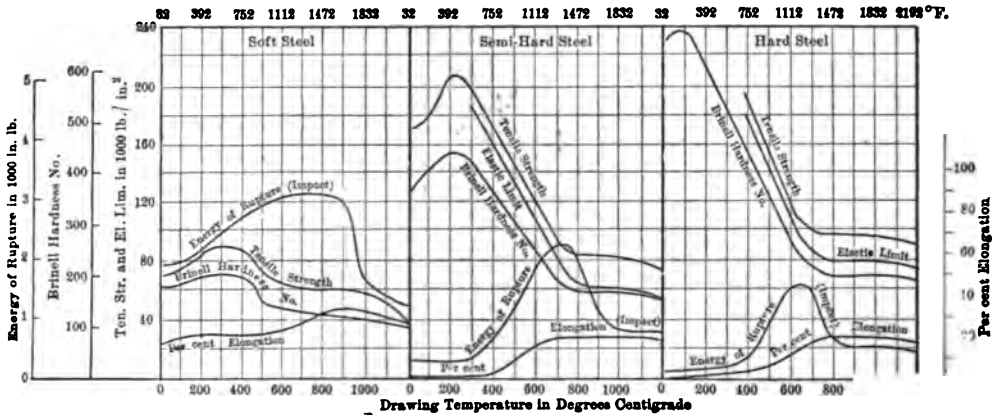


FIG. 32.—Effects of Drawing Temperature on the Mechanical Properties of Three Grades of Steel. (Grard.) Impact tests were made on notched specimens.

For drawing temperatures below  $400^{\circ}\text{C}$ . the energy of rupture the semi-hard and hard steels is low and constant. Again when annealing temperatures are reached in the drawing process t

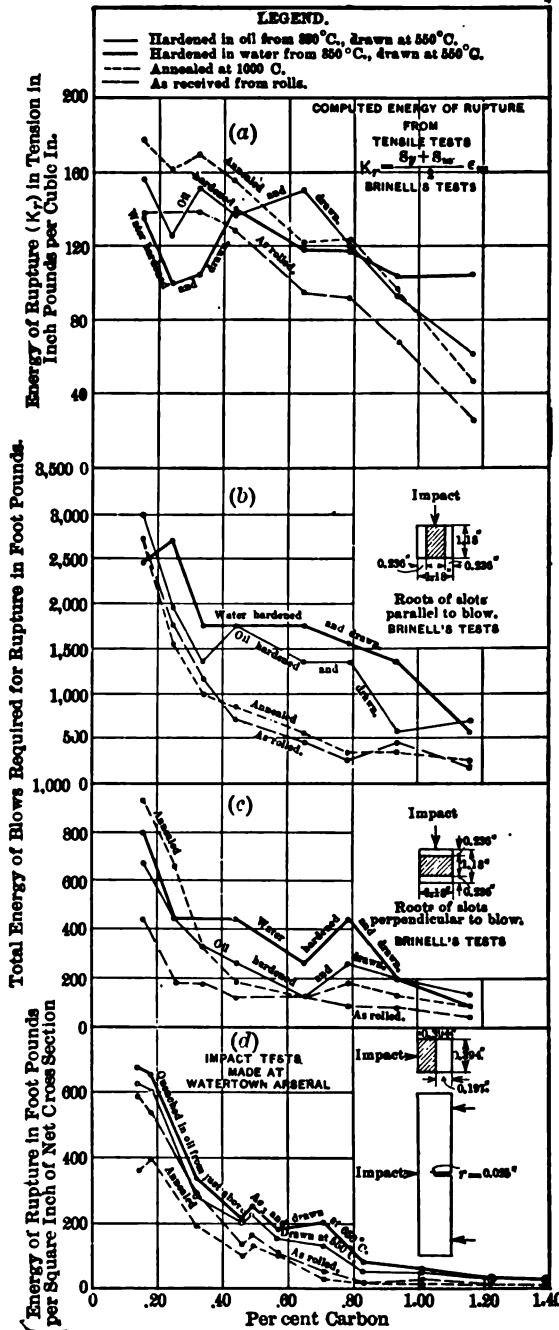


FIG. 33.—The Influence of Carbon on the Toughness of Steel as Measured by the Energy of Rupture in Tensile Tests and in Impact Tests.

ness abruptly decreases. It appears, therefore, that the quenched steels in the martensitic state, and those which have been tempered into a troostitic state are very strong and hard but they do not possess the toughness of the sorbitic steels formed by drawing at higher temperatures (500° to 700° C.). Furthermore, the pearlitic steels formed on annealing after quenching, although more ductile, have less shock resistance than those in the sorbitic state.

Results of some of Brinell's impact tests have been plotted in Fig. 33 (b) and (c). These also indicate the superior resistance of low-carbon steels under impact and show that the medium and high-carbon steels are materially improved by quenching and drawing at a high temperature. They do not show, however, that such treatment is so beneficial for the low-carbon steels.

By comparing Fig. 33 (a) with Fig. 33 (b) and (c) it becomes evident that the energy of rupture computed from tension tests is not a

measure of the efficiency of a heat treatment in toughening steels. In other words the efficiency of a toughening treatment should be judged by the results of impact tests. In general the nicked impact test with the blow applied normal to the root of the nick is the most severe type of test for such purpose.

In Fig. 33 (d) are plotted results of Charpy impact tests made on the same steels as used by Nead (see Fig. 31). The results are more consistent than those of Brinell and, likewise, show the beneficial effects of drawing oil-quenched steels at 650° C., thus producing a fine-grained sorbitic structure giving maximum toughness.

### EFFECTS OF MECHANICAL WORK

**712. Effect of Hot Work on Structure.**—The mechanical working of steel may be hot or cold; it may be carried out at temperatures above or below the transition zone. In heating steel above the critical temperature, there is a complete obliteration of existing grain, and a progressive increase in grain size of the newly formed austenite with temperature rise above the critical range, and with time intervals which will allow of normal crystallization. Symmetry of crystallization is the tendency in the austenitic zone, and the size and character of grain thus attained is not destroyed during the reverse transition in cooling to normal temperature.

Mechanical work results in distortion of the grains causing flattening in the direction of the pressure and consequent destruction of the normal symmetry. Within the austenitic range the metal will split up into a mass of grains of symmetrical character, and approximately of the dimensions determined by the distortion; these will in turn grow into an aggregate of larger symmetrical grains conforming to the temperature and time conditions. If, therefore, as is usual in mechanical work operations, distortion and reduction of grain size within the austenitic temperature range is accompanied by a gradual decrease of temperature of the metal, the resultant grain must be finer than the original, since there has been mechanical reduction, accompanied by a lower end temperature which will largely determine the size of the final grain. This size will be a function of the amount of reduction of the initial grain by the mechanical operations, and the finishing temperature as compared with the critical. Heavy reductions will tend to result in a fine-grained structure; this in turn will tend to increase in size the higher the finishing temperature is above the critical one, and the more the time approaches that needed for complete aggregation. From the standpoint of the doctrine of grain size—that strength and ductility are the accompaniment of fineness of structure—

the importance of hot work in improving the physical quality of steel is obvious, and it is especially beneficial if the reductions are heavy and at finishing temperatures as close as practicable to the critical zone. A comparison of Figs. 11 and 12 shows the marked improvement in refining the grain which has been accomplished by forging a piece of cast steel.

The specifications of the United States Navy for steel forgings demand that the diameter of the grains in the finished part, as determined under the microscope, shall not exceed 0.005 in.

**713. Effects of Hot Work on Properties of Steel and Iron.**—It is evident that the rate of cooling of a steel section will have an important

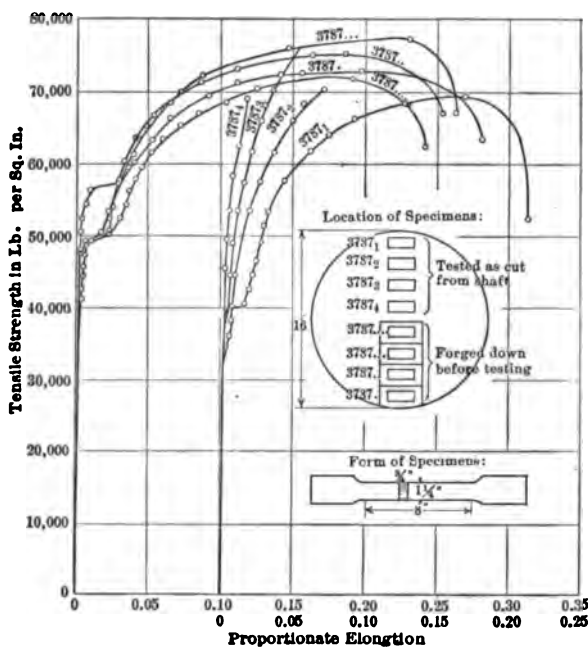


FIG. 34.—Showing the Varying Character of Metal in Different Parts of the Cross-section of a Large Steel Shaft when Forged under a Ten-ton Hammer. (*Tests of Metals*, 1885.)

influence upon the structure and physical properties, provided the mechanical work is finished at a temperature above the critical range. That is, in addition to the direct effect of mechanical refining of the grain, there is the accompaniment of the effects of hardening and tempering, because of direct treatment or the equivalent result due to natural cooling conditions. The tempering effects conform to the principles set forth heretofore, and are proportionately greater in high-carbon or special-alloy steels than in the mild steels which are of greatest importance in usual construction. The best results are obtained by the proper correlation of mechanical work and heat treatment of a steel of suitable chemical composition, all determined by consideration of the service conditions. A few examples illustrating the effects of hot working follow.

In Fig. 34 is shown the cross-section of a steel shaft 16 in. in diameter (which broke soon after being put in service) from which eight

important influence upon the structure and physical properties, provided the mechanical work is finished at a temperature above the critical range. That is, in addition to the direct effect of mechanical refining of the grain, there is the accompaniment of the effects of hardening and tempering, because of direct treatment or the equivalent result due to natural cooling conditions. The tempering effects conform to the principles set forth heretofore, and are proportionately greater in high-carbon or special-alloy steels than in the mild steels which are of

test-specimens were cut, lying symmetrically in a diametral section as shown. Four of these were tested as cut from the shaft. The other four were forged down after cutting out. The plotted results show: The elongation of the unforged specimens varied from 21 per cent in the specimen taken from near the surface of the shaft to 2 per cent in the specimen coming from near the center. In the forged specimens, however, taken from the opposite side of the disc, the elongation varied from 28 per cent near the surface of the shaft to 24 per cent near the center, thus showing that the material was identical throughout when it had been similarly worked. In other words, the material

near the center of the shaft was in its primitive condition when first cast, while that near the surface was that of well-rolled steel. This shows the necessity of forging large shafts under enormously heavy hammers, or, better, the necessity of using only hollow-forged shafts for such service.

The amount of reduction which a piece receives in rolling materially influences the mechanical properties. Bullens \* cites tests of bars ranging from  $\frac{1}{2}$  in. to 3 in. in diameter all rolled from the same ingot. The tensile strengths of these bars varied from 137,000 to 100,000 lb. per square inch, respectively.

In Fig. 35 the characteristic influence of thickness on the properties of mild steel bars is well shown. It will be observed that the strength and elastic ratio decrease slightly but the ductility increases as the thickness of the metal is increased. Annealing greatly reduces these effects.

From Fig. 36 it may be seen that the variation in ultimate strength and in the elastic limit for various thicknesses of metal is much greater when the metal leaves the

rolls at a dull red heat. Working at this temperature slightly increases the ultimate strength and ductility and raises the elastic limit from 8 to 10 per cent above the values gotten at normal finishing temperatures (a bright red). After annealing, Campbell's tests show that specimens finished at a dull red heat still have properties superior to normal specimens similarly annealed.

In general the apparent elastic limit rises as the thickness of section

\* *Steel and Its Heat Treatment*, p. 229.

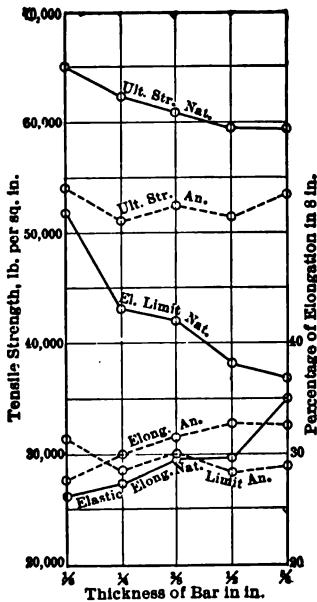


FIG. 35.—Effect of Thickness on the Mechanical Properties of Mild Steel, Natural and Annealed. (Campbell's *M'fg and Prop. of Iron and Steel*.)



diminishes. Since steel columns are built from comparatively thin sections of metal (generally from  $\frac{1}{4}$  to  $\frac{1}{2}$  in. in thickness), and since the ultimate strength of these is dependent wholly on the apparent elastic limit, and not at all upon the ultimate strength, it is necessary to evaluate this elastic limit for the particular thicknesses of sections

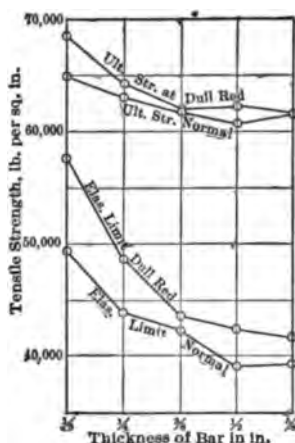


FIG. 36.—Influence of Thickness on Mechanical Properties when the Percentage of Reduction in Rolling is Constant, Last Passage in Rolls being at Normal and Dull Red Heat, Respectively. (Campbell's *Manufacture and Properties of Iron and Steel*.)

used, rather than from special test bars, which are usually not less than  $\frac{3}{4}$  in. in thickness.

Differences in the amount and character of work which various parts of a steel shape receive during rolling often cause considerable discrepancies in mechanical properties. From Fig. 3, Ch. XIX, it will be noted that the material near the center of the web and in outer portions of the flanges of an I-beam is worked more in rolling than the portions at the junction of the flange and web. Results in Table 5 show variations in the properties of test-pieces cut from these parts of several I-beams.\*

**714. Methods of Cold Working.**—Cold working of steel, or reduction of area at temperatures well below the critical zone, results in distortion of the grain in accordance with the applied forces, and this distortion remains because of lack of mobility or tendency to acquire symmetry at temperatures much below the critical, Fig. 37. Cold distortion causes increased density and produces an internally strained condition in the steel. It

is accompanied by increased hardness, tensile strength, and, if a period of rest is allowed, by an increase in elastic limit.† The ductility and shock resistance, however, are lowered by cold working. Cold drawing, cold rolling, cold pressing (or flanging), and cold twisting are the usual methods of cold working metals. The drawing and rolling processes are very effective in securing parts of exact cross-sectional dimensions. The cold-drawing process is used mostly on pieces of small cross-sectional area,—such as wire, automatic screw stock, small shafting and tubing,—where a large reduction in area can be made in a single draft. For large shafts and axles cold rolling is more efficient than cold drawing. Cold rolling impairs the ductility of the metal less than cold drawing, probably

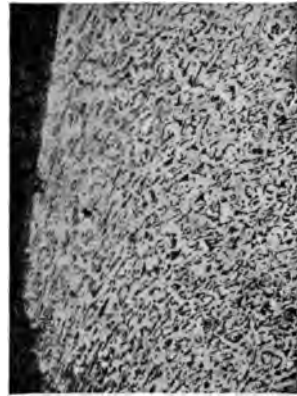
\* From the carefully conducted tests of E. L. Hancock, *Proc. A. S. T. M.*, Vol. 10, p. 248.

† Elastic limit is used synonymously with proportional limit in Arts. 714 to 717.



(a) The distortion is much less in the hot-rolled metal (7) than in the cold-rolled (8).

← Motion of Punch.



(b) Distortion due to cold punching. Hole at left.

FIG. 37.—Distortion Due to Cold Work. (Mag. = 50d.)

TABLE 5.—TENSION TESTS OF STEEL SPECIMENS CUT FROM I-BEAMS. (HANCOCK).

The specimens were taken from the center of the web, from the flange near the web, and from the root of flange and web. Elastic limit was determined by an extensometer reading to 0.0001 inch.

Depth of Beam (in.).	Weight per Foot (lb.).	ELASTIC LIMIT (lb./in. <sup>2</sup> )			MAXIMUM STRENGTH (lb./in. <sup>2</sup> )			PER CENT ELONGATION IN 8 IN.		
		Web.	Flange.	Root.	Web.	Flange.	Root.	Web.	Flange.	Root.
STANDARD BEAMS										
6	13.5	32,000	34,000	34,000	68,300	65,300	64,300	18.7	29	29
10	25	30,000	25,500	33,000	56,500	54,600	58,600	28.3	28.1	25
10	25	24,750	26,600	27,000	72,300	61,200	66,400	20.2	34.4	25
12	31.5	29,500	22,500	19,500	61,800	59,500	63,700	31.3	26.6	25
12	40	31,500	24,000	19,500	59,800	62,500	61,500	31.2	21.9	31.2
12	40	28,500	18,700	24,000	62,200	62,200	61,700	30	18.7	31
15	42	31,500	29,900	22,500	68,700	.....	56,000	28.2	....	26.6
15	60	19,500	24,000	16,500	53,300	55,200	53,800	29	29	29
15	80	27,000	19,500	19,500	54,300	55,300	55,000	36	31	33
18	55	31,500	25,500	19,500	55,800	56,700	60,400	28.5	29	28.5
20	65	34,500	22,500	13,500	62,700	63,800	64,000	28.5	28	30
BETHLEHEM H-BEAMS										
24	73	33,000	24,000	22,500	72,600	70,000	67,300	25	28	28.1
18	48.5	19,500	21,000	28,500	59,600	57,300	56,200	26.6	31.3	31.3
12	36	31,500	33,000	24,000	72,600	66,000	68,400	21.9	26.6	25
Mean Values		28,875	25,050	23,107	62,893	60,738	61,236	27.39	27.82	28.40
Based on Web		1.00	.865	.800	1.00	.965	.974	1.00	1.018	1.039

because the total reduction in area is effected by a large number of light passes through the rolls. The reductions in either of these processes are very much less than in rolling or drawing hot. Cold pressing is much used in fashioning thin parts in car construction, and in making of sheet metal forms. Cold-pressed parts which must withstand shock require annealing. Cold twisting is used to raise the elastic limit and ultimate strength of square steel bars for reinforcing concrete.

Cold punching of holes in boiler plate, in structural steel and forms made of wrought iron and soft steel is very commonly practiced. The method is rapid and cheap but causes heavy distortion in the vicinity of the holes as shown in Fig. 37*b*. Since the metal on the die side of the hole is considerably embrittled by this process (see Fig. 16, Ch. III), and is also rendered more susceptible to corrosion (see Fig. 1, Ch. XXIX), good practice demands that the holes be punched about  $\frac{1}{8}$  in. undersize and reamed to required dimensions.

**715. Effects of Cold Work on Properties.**—Tests on hot-rolled and cold-rolled shafting by Thurston at Cornell University\* show that cold rolling increases the elastic limit in tension from 15 to 97 per cent, the tensile strength from 20 to 45 per cent. Under cross bending the elastic limit is raised 11 to 30 per cent and the yield point 13 to 69 per cent. The elastic limit in torsion is raised from 28 to 40 per cent. That the effect of cold rolling on the strength penetrates undiminished to the axis of the bar is demonstrated by a portion of Thurston's results, given in Table 6. The ductility of the metal at the surface is greater than that at the center but the discrepancy is most marked in the cold-rolled metal. In elastic resilience the cold-rolled metal is superior to the hot-rolled; in energy of rupture it is inferior to the hot-rolled metal. The modulus of elasticity is slightly increased by cold rolling, in these tests by about 1,000,000 lb. per square inch.

From a series of tests by A. J. Wood† on cold-drawn steels of low-carbon content the following conclusions seems justified. Cold drawing by reducing the diameter about  $\frac{1}{16}$  in. in one draft increases the maximum strength of bars less than  $1\frac{1}{2}$  in. in diameter about 10 to 15 per cent. It increases the elastic limit about 100 per cent, reduces the elongation by 75 per cent and the reduction in area by about 15 per cent. Cold drawing raises the strength and elastic limit both by direct stretching of the metal and by lateral compression which also causes lengthening of the piece. The properties of the metal are approximately uniform throughout the piece.

\* See abstract in *Machinery*, April, 1917.

† *Engr. News*, Vol. 59, p. 63.

TABLE 6.—RESULTS OF TESTS ON COLD-ROLLED AND HOT-ROLLED STEEL BARS.

Diameter of Bar in Inches.	Stress in Pounds per Square Inch.				Elongation in 8 In. in Per cent.	Reduction of Area in Per cent.	Resilience in Inch Pounds per Cubic Inch.		Modulus of Elasticity in 1000 Lb. per Square Inch.
	Elastic Limit.	Yield Point.	Maximum.	Breaking.			Elastic Limit.	Breaking.	
COLD-ROLLED STEEL									
0.349	54,000	62,500	73,637	51,613	7.50	53.70	48.14	5,600	30,393
0.526	59,500	66,000	75,739	55,060	10.12	54.10	56.21	7,340	31,560
0.772	53,500	66,000	76,636	54,548	12.31	55.70	47.02	9,000	30,476
1.019	52,000	62,000	73,778	50,205	14.87	57.80	42.25	10,480	30,056
HOT-ROLLED STEEL									
0.355	30,000	35,250	58,606	40,164	22.06	68.0	15.32	10,900	29,418
0.509	31,900	35,500	58,809	41,569	24.31	61.7	17.51	12,860	29,073
0.754	29,750	35,750	59,747	41,793	29.25	64.2	15.67	15,280	28,337
1.010	30,250	35,250	61,210	43,392	30.00	63.8	14.43	16,740	29,126

In the above tests by Thurston and by Wood the properties were determined from pieces turned down to various proportions of the original diameter of the rolled section, but no direct measurements of the relief of internal stress due to the machining are reported. From the experiments of Heyn \* and others, however, it seems certain that cold-worked metals, especially those which are cold drawn, are liable to severe internal stress. In a cold-drawn piece the core is placed under compression and the outer layers under tension. Thus, in one of the Heyn tests on a steel bar (Ni=25, C=0.3 per cent) which was reduced in diameter from 1.34 to 1.22 in. by cold drawing, the maximum tensile stress near the surface of the bar was 50,000 lb. per square inch and the maximum compressive stress along the axis amounted to 54,000 lb. per square inch. By annealing at 850° C. the maximum internal stress was made less than 3500 lb. per square inch. Heyn also cites instances of failures of cold-drawn parts of steel and brass caused by internal stress.

From the foregoing it appears that a material increase in elastic limit and ultimate strength of steel with a corresponding loss in ductility and toughness may be secured by light reductions in cold draw-

\* *The Jour. Inst. of Metals*, Vol. 12, p. 12.

ing, but if the reductions are large the internal stresses thus induced may adversely effect the strength as well as toughness and ductility.

The effects of cold twisting on the strength and ductility of mild

steel bars is illustrated in Fig. 38.\* The elastic limit in these tests was increased from 10.5 per cent for the  $\frac{3}{8}$ -in. bars twisted one turn in 12d to 80.5 per cent for the  $\frac{3}{8}$ -in. bars twisted  $3\frac{1}{4}$  turns in 12d. For manufacturing twisted reinforcing bars one turn in a length of 6 to 12 diameters is commonly used. In a paper before the A. S. T. M., J. J. Shuman, a manufacturer, advocated 1 twist in 8 to 10 diameters for Bessemer steel of 60,000 lb. per square inch tenacity and 1 turn in 5 to 7 diameters for basic open-hearth material of like strength. The data in Fig. 38 indicate that twisting a bar of mild steel through one turn in 8 diameters causes an increase in strength of 13 to 22 per cent but produces a decrease of 50 to 70 per cent in elongation. Under such treatment the elastic limit will be raised 40 to 60 per cent.

It becomes apparent that cold twisting produces a large increase in elastic ratio and a considerable increase in strength. These changes are accompanied by a marked reduction in ductility and a probable loss in shock resistance.

**716. Relief of Distortion Due to Cold Working.**—In wire drawing,

\* Abstract in *Jour. Am. Soc., M. E.*, Dec., 1916, of tests by Whitney and Dohner at University of Colorado.

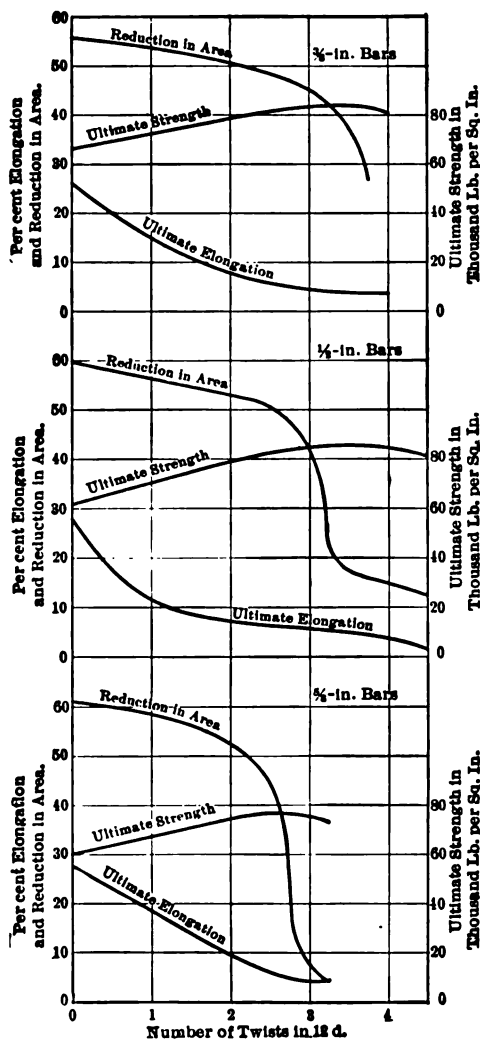
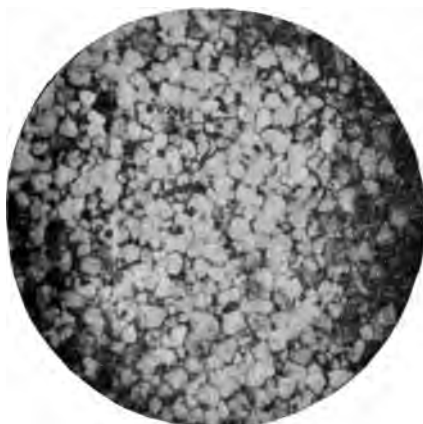


FIG. 38.—The Effect of Cold Twisting on the Strength and Ductility of Mild Steel Bars. (Whitney and Dohner.)

several passes through the dies makes the steel so brittle that annealing is necessary to restore ductility and ensure further reduction without rupture.

It is usually considered that temperatures above the critical are necessary to remove the distortion of structure due to cold work.



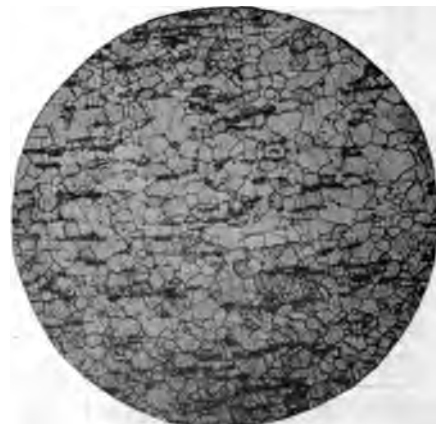
(a) Steel rod annealed.



(b) Steel wire after one draft. Area of rod reduced 15%.



(c) Steel wire after several drafts. Area of rod reduced 60%.



(d) Steel wire hard-drawn and then annealed at a temperature below critical range.

FIG. 39.—Effects of Drawing and Annealing on the Structure of Soft Steel Wire (0.08% C.). Magnification 95 diameters. (J. F. Tinsley.)

However, in the manufacture of wire, the annealing temperatures (process annealing) are approximately  $600^{\circ}\text{C.}$  or well below the  $A_{c1}$  point. That there is effective removal of grain distortion and restoration of ductility is indicated by Figs. 39a to 39c and Table 7, representing typical practice in the drawing and annealing of mild steel wire

of 0.10 per cent carbon content. (J. F. Tinsley before Am. Ir. and St. Inst., May, 1914).

Corresponding results in drawing and patenting 0.50 carbon steel wire are given in Table 8. The patenting operation consists in drawing the wire through a heated tube, at a speed and temperature regulated according to the carbon content and size of the rod and wire; then cooling it in the air or in a bath of molten lead, depending upon the final structure and properties desired. The process is in effect one of combined annealing and tempering, and serves to obliterate the distortions of drawing and to give to the finished product the required combination of strength and toughness, mainly by preventing coarse

TABLE 7.—THE EFFECT OF COLD DRAWING AND ANNEALING ON THE PHYSICAL PROPERTIES OF 0.10% CARBON STEEL. (TINSLEY)

Condition of Material.	Percent Reduced in Drawing.	Tensile Strength lb./in. <sup>2</sup> .	Per cent Elongation in 10 in.
Green Rod. .... No. 5 gage, <i>d</i> = .207 in.	0	68,000	25
First draft. ....	47	113,000	2½
Third draft. ....	82	150,000	1½
Process annealed	0	60,000	30
First draft. ....	35	93,000	3
Third draft. ....	70	130,000	2
Fifth draft. ....	85	145,000	1½
Process annealed	0	62,500	28
First draft. ....	21	82,000	3½
Third draft. ....	62	124,000	2
Fifth draft. ....	80½	143,000	1½
Seventh draft. ....	88½	151,500	1
Annealed. ....	0	68,000	25
Dead soft annealed after first and second annealings	....	50,000	32

TABLE 8.—THE EFFECT OF COLD DRAWING AND PATENTING ON THE PHYSICAL PROPERTIES OF 0.50% CARBON STEEL. (TINSLEY)

Condition of Material.	Per cent Reduced in Drawing.	Tensile Strength lb./in. <sup>2</sup> .	Per cent Elongation in 10 in.
Green Rod. .... No. 5 gage, <i>d</i> = .207 in.	0	95,000	10.0
First draft. ....	28½	122,000	2.9
Second draft. ....	51	146,000	2.8
Patented. ....	0	115,000	8.2
First draft. ....	30	143,000	2.8
Third draft. ....	50½	163,000	2.7
Fourth draft. ....	65	176,000	2.6
Patented. ....	0	128,000	7.8
First draft. ....	30	156,000	2.0
Third draft. ....	66	190,000	1.9
Fourth draft. ....	76	208,000	1.8
Patented. ....	0	156,000	6.0
First draft. ....	30	184,000	2.0
Third draft. ....	66	218,000	1.9
Annealed after first patenting.	....	70,000	18.0

coalescence of the pearlitic constituents. It is chiefly adapted to medium- or high-carbon steel products such as piano wire or wire for rope strands.

**717. Effects of Overstrain in General.**—The various methods of cold working, previously discussed, all stress the metal beyond its elastic limit and each produces a particular kind of overstrain. In a general way it has been shown that overstraining a bar in a certain direction, say in tension, will raise the yield point to the overstraining load and will slightly increase the ultimate strength. The magnitude of the effect on the ultimate will depend upon the amount of overstrain. The limit of proportionality is greatly lowered immediately after overstrain.\* There is, however, more or less complete recovery as time elapses, and the proportional limit may be eventually raised above the normal value. If the overstrained bar is put under the opposite kind of stress (compression) the yield point and proportional limit of the bar are lowered, they may even vanish if the period of rest after overstrain is small. (See Fig. 6, Ch. XXI.) There is also evidence to show that the effects of overstrain are felt in all directions. Howe † has shown that the ball hardness of a tensilely overstrained steel bar is increased in all directions, in other words, that the effects of simple overstrain are isotropic in hardening the metal. Evidence of influence of overstrain in directions inclined to the overstrain axis is also afforded by Table 9.

From these results it appears that the effect of tensile overstrain on tensile properties is greatest in the direction of the overstrain and least at right angles to it. On the other hand, tensile overstrain effects the greatest increase in compressive elastic limit and strength in directions normal to the overstrain. (See also Art. 671.)

**718. Grain Growth in Overstrained Metal.**—When a pure metal or alloy of solid solubility is heated subsequent to overstrain a growth in the grain may be produced at temperatures below the critical range. Thus very low-carbon steel, or ingot iron, exhibits a decided growth of its crystal grains after overstrain, on heating to a temperature which varies inversely with the magnitude of the overstrain. For example, McAdam ‡ in experimenting on ingot iron containing 0.034 per cent carbon found that this metal when reduced 30 per cent in thickness began to recrystallize when heated for  $8\frac{1}{2}$  hours at  $900^{\circ}$  F. ( $482^{\circ}$  C.), when reduced 15 per cent crystallization commenced at  $1000^{\circ}$  F., and when reduced 10 per cent crystallization began at  $1100^{\circ}$  F. In no case, however, did coalescence of grains take place in the recrystallized

\* See *Tests of Metals*, 1915, p. 127.

† See Howe's paper on simple overstrain in *Proc. A. S. T. M.*, Vol. 14, p. 8.

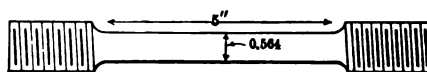
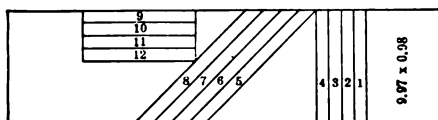
‡ *Proc. A. S. T. M.*, Vol. 17, p. 59.



material until the temperature of annealing was raised above 1475° F. (801° C.).

TABLE 9.—COMPARISON OF TENSION AND COMPRESSION TESTS ON ANNEALED AND UNANNEALED STEEL BARS OF IDENTICAL MATERIAL WHICH HAD BEEN STRESSED BEYOND ITS ELASTIC LIMIT.\*

TENSION TESTS.								COMPRESSION TESTS.			
Marks.	How Taken.	Condition.	Elastic Limit per Square Inch.	Ultimate Strength per Square Inch.	Elongation in 5 In.	Contraction of Area.	E in 1,000,000 lb./in. <sup>2</sup> .	Marks.	Elastic Limit per Square Inch.	Ultimate Strength per Square Inch.	E in 1,000,000 lb./in. <sup>2</sup> .
1	Crosswise	Unannealed	40,000	66,320	12.0	24.6	30.2	3	51,000	58,100	34.3
2	"	Annealed	40,000	63,160	22.0	33.5	31.2	4	43,000	43,000	33.0
5	Diagonally	Unannealed	40,000	66,760	16.0	59.3	30.2	7	47,000	54,380	34.3
6	"	Annealed	42,000	62,840	27.5	61.5	30.7	8	40,000	43,000	31.8
9	Lengthwise	Unannealed	63,000	73,000	18.0	61.5	31.2	11	35,000	48,940	34.9
10	"	Annealed	47,000	64,000	26.0	61.5	30.7	12	42,000	45,000	32.4



Tension Specimen

\* These specimens were cut from an eye bar which had been stressed to 54,350 lb. per square inch, 3 years and 3 mos. before these tests were made. The original elastic limit was 34,400 lb. per square inch. The position of the specimens in the original bar is shown in the figure above. The compression specimens were approximately 0.92 in. in diameter. (From *Tests of Metals*, 1890, p. 731.)

The presence of impurities such as slag in wrought iron or carbon, in the form of pearlite, in steel appear to prevent grain growth. Consequently wrought iron and steel containing over 0.15 per cent carbon are not subject to grain growth.

Inasmuch as coarse grain makes the metal very brittle the phenomenon of grain growth bears an important relation to the properties of susceptible metals when worked in the cold and subsequently reheated. The phenomenon is often called "Stead's Brittleness," after J. E. Stead, who first investigated it.

Brittleness is also caused in low-carbon steel and wrought iron by prolonged working at a blue heat (225–300° C.). It appears quite likely that the crystal growth so produced originates from causes similar to those producing "Stead's Brittleness."

## INFLUENCE OF FORM ON PROPERTIES

**719. The effect of an abrupt contraction in cross-section of a bar under tension or bending is to cause high concentration of stress at the periphery of the groove or notch; also the shoulders of the groove or notch prevent free elongation of the metal at the contracted section and increase its strength.** Fig. 40 shows the effects of various types of grooves on the distribution of stress in rectangular bars under tensile stress. The yield point of the iron used was 36,900 and the ultimate strength 56,500 lb. per square inch. The mean tensile stress in each case was 10,650 lb. per square inch. It will be noted that the calculated stresses at the root of the V-shaped notches exceeded the yield point of the metal.

The superior resistance per unit area of the steel at root of groove as compared with steel in uniform bars is shown by the tests of M. Duguet on hard steel and M. Barba on soft steel which are plotted in Fig. 41.

**720. The Influence of the Form of the Thread on the Strength of Screw-bolts.**—This subject has been investigated by Prof. Martens,\* and his results are here given.

Two grades of mild steel were used for these bolts, all of which were cut from round bars originally 35 mm. (1.4 in.) in diameter. The softer material,

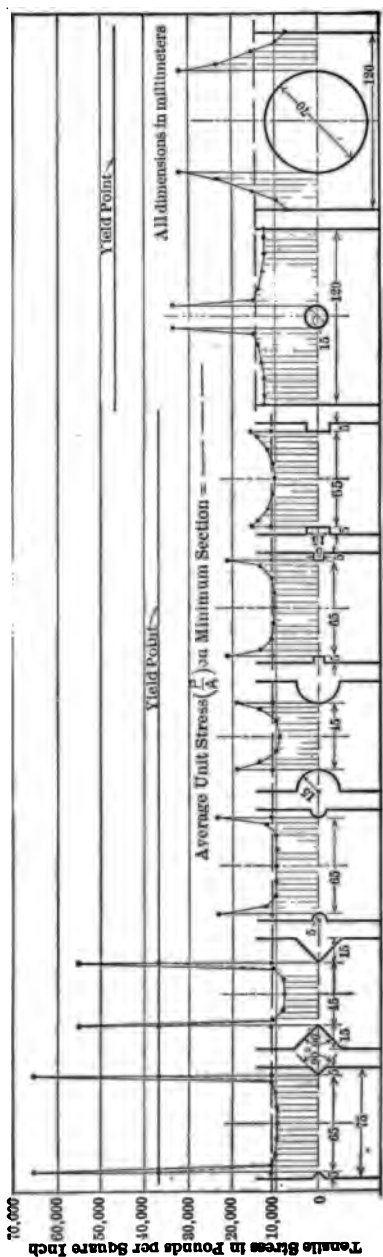


Fig. 40.—The Effect of Abrupt Changes in Rectangular Cross-sections on the Distribution of Tensile Stress. (E. Preuss, *Zeit. Vereines deutscher Ing.*, 1913, Pt. 1, p. 664; *Mittel. über Forschungsarbeiten auf dem Geb. des Ingw.*, Vol. 128, p. 54.)

\* At the request of the German Society of Civil Engineers. The results were published in *Zeits. d. Ver. deutsch. Ing.* for April 27, 1896. The abstract here given was made by J. B. Johnson and published in the *Digest of Physical Tests* for July, 1896.

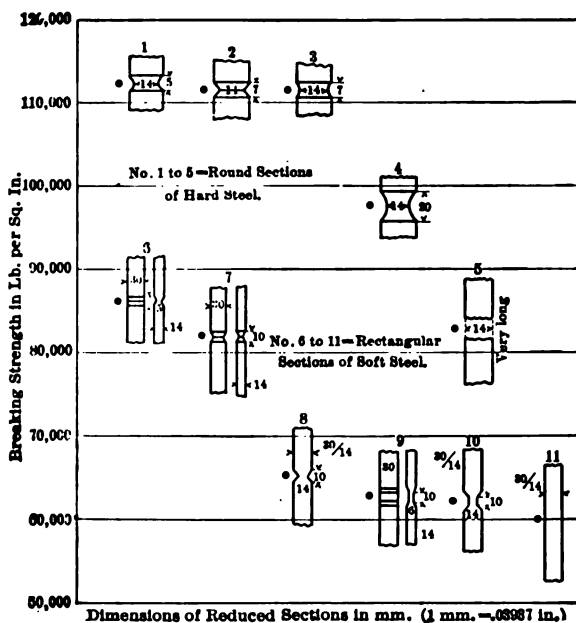






FIG. 41.—Effect of the Form of the Reduced Cross-section on the Tensile Strength of Two Kinds of Steel. (*French Com. Rep.*, Vol. 3, p. 40.)

having a tensile strength of 53,500 lb. per square inch, was used for screw-bolts approximately 1 in. in diameter, and the harder material, having a tensile strength of 62,000 lb. per square inch, was used for the screw-bolts, which were reduced to approximately  $\frac{1}{2}$  in. in diameter. Four such bolts were made of each of these sizes for each of the four styles of thread shown in Table 10, making in all 32 bolts with screw-threads which were tested. Two of each of these sets were tested in plain tension, the pulling force being applied to the inner face of the nut at one end, and increased until rupture occurred. The outer two bolts of each set were tested also in tension, but

TABLE 10.—ABSOLUTE AND RELATIVE STRENGTHS OF THREADED BOLTS IN POUNDS PER SQUARE INCH. (MARTENS)

Kind of Thread.	Form of Base of Thread.	Diameter, In.	STRESS APPLIED BY			PROPORTION.				
			Machine.		Nut. $S_m$	Test Bar = 100.			$S_g = 100.$	
			G.* $S_g$	T.* $S_t$		$S_g$	$S_t$	$S_m$	$\frac{S_t}{S_g}$	$\frac{S_t}{S_m}$
(a) Sharp.....		1 $\frac{1}{2}$	62,430 71,100	61,580 70,400	49,920 62,720	116.8 114.9	115.2 114.0	93.4 101.4	98.9 99.2	80.0 88.2
(b) Whitworth.....		1 $\frac{1}{2}$	62,160 67,000	61,300 69,400	44,800 58,880	116.2 109.4	114.6 112.2	83.8 95.2	98.9 100.3	72.2 86.9
(c) Sellers.....		1 $\frac{1}{2}$	60,300 70,250	60,020 68,120	52,330 62,720	112.8 113.6	112.2 110.1	97.9 94.5	99.6 96.9	86.9 83.2
(d) German Soc. of Engr.....		1 $\frac{1}{2}$	60,730 69,260	61,160 73,670	47,640 62,720	113.6 112.0	114.4 119.1	89.1 101.4	100.7 106.4	78.6 90.8
(e) Normal test bar.		1 $\frac{1}{2}$	..... .....	53,480 61,860	..... .....	..... .....	..... .....	100 100		

\* G. = grooved; T. = threaded.

under a torsional action resulting from the continuous turning of the nut as the load increased to rupture. In this case the distortion resulting from the permanent elongation of the bolt was nearly all taken up by the movements of the testing-machine, the distortion taken up by the turning of the nut being the least possible to maintain a continuous torsional action at this point.

The same bars were also tested as plain tension-test specimens with cylindrical bodies, and again with grooves turned into them of the same shape as the corresponding screw-threads, leaving the same diameter at the bottom of the groove as obtained at the base of the threads. The actual and comparative average results of all of these tests are given in the table, from which the following conclusions may be drawn:

1. When subjected to plain tension both the screw-threads and the grooved sections were stronger than the plain bars of the same net area of cross-section, this excess of strength having an average value of about 14 per cent.

2. There is no very marked difference in the average strength of the bolts on which the several styles of thread were cut, the perfectly sharp groove shown at (a) in table being slightly stronger than the others.

3. The weakening effect of the turning of the nut under stress at rupture is much less than might have been predicted, when the distortion of the screw below the nut by permanent elongation is taken into consideration. The tests indicate for this case a strength of the 1-in. bolts about 20 per cent less than that of the plain bars, and of the  $\frac{1}{2}$ -in. bolts about 15 per cent less than that of the plain bars.

4. In general it may be said that the turning of the nut upon the bolt at rupture reduces the strength of the net section of the bolt by about 30 per cent.

5. It is very probable that the four forms of screw-threads here shown would show very different results under fatigue tests from repeated stresses, and also for static loads on high-carbon steel. Under repeated loads and under shock it is probable that the sharp re-entrant angle of thread (a) would develop incipient cracks much earlier than either of the other forms, and that probably the Whitworth thread (b) would be the last to develop this kind of weakness, either with soft metal under repeated loads or with high-carbon steel under static loads.

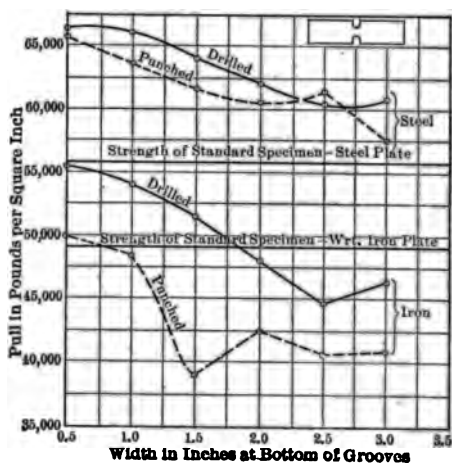


FIG. 42.—Variation in Strength of  $\frac{1}{2}$ -in. Plate for Varying Widths at Bottom of Groove. Each point represents 3 to 8 tests. (*Tests of Metals*, 1882.)

**721. The Tensile Strength of Grooved Plates** is a measure of the tensile strength of a riveted joint when failure occurs by tearing the plate. This strength is found to be a function of the width of the net section at the bottom of the groove, as well as of the method of making the hole, and of the character of the material. These effects are all shown in Fig. 42 for  $\frac{1}{2}$ -in. plates of wrought iron and of 56,000-

lb. steel. The steel, being more ductile, is stronger in the grooved than in the plain (standard) section, while the reverse is the case with

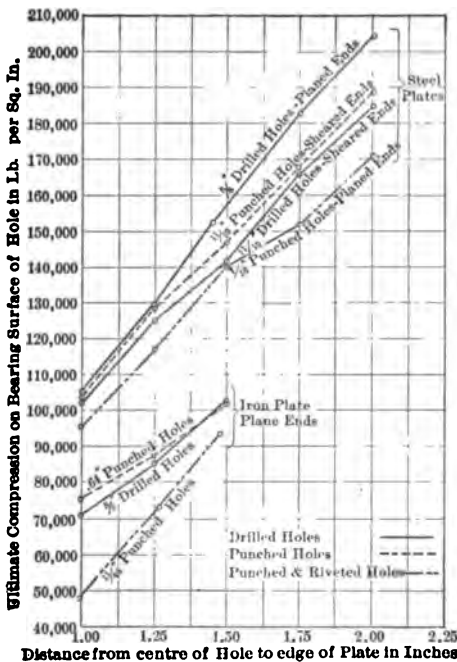


FIG. 43.—Bearing Resistance on Rivet Holes at Rupture when Failure Occurs by Tearing Out of Hole. Tensile strength of steel plates = 60,000 lb. per sq. in. (*Tests of Metals*, 1882.)

wrought iron, except with drilled specimens, where the width of the net section was less than  $1\frac{3}{4}$  n.

722. The bearing resistance of steel and iron plates is shown in Fig. 43. This is seen to increase directly with the distance of the hole from the edge of the plate. When this distance agrees with ordinary practice the resistance is so high that it would seem a working bearing stress of 16,000 lb. per square inch might be employed for iron, and of 24,000 lb. per square inch for steel plates. The stresses here plotted were the bearing stresses at rupture, where the plates had so reduced in thickness as to destroy all frictional resistance. Much more, then, could high working stresses be employed, since for these the frictional resistance is very great. It

appears that the ordinary rules for proportioning riveted joints might well be modified so as to allow higher bearing stresses, especially on steel. With wrought iron, especially when the stress is transverse to the fiber, more care must be exercised, as this material is liable to be very weak in this direction.

723. Resistance of Steel when the Compressed Area is Confined Laterally.—When a compressive load is applied over the full cross-section of a short prism or cylinder

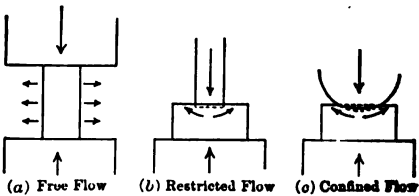


FIG. 44.

as in Fig. 44a the metal is free to flow laterally at all cross-sections excepting those in close proximity to the ends. If the load is uniformly applied over a portion of the surface as in Fig. 44b the metal under the loading surface must flow laterally against a ring of unstressed metal.

The flow is therefore restricted and the elastic limit will be higher than in the first case. Again, if Fig. 44c represents conditions of loading, Direction along the Rail.

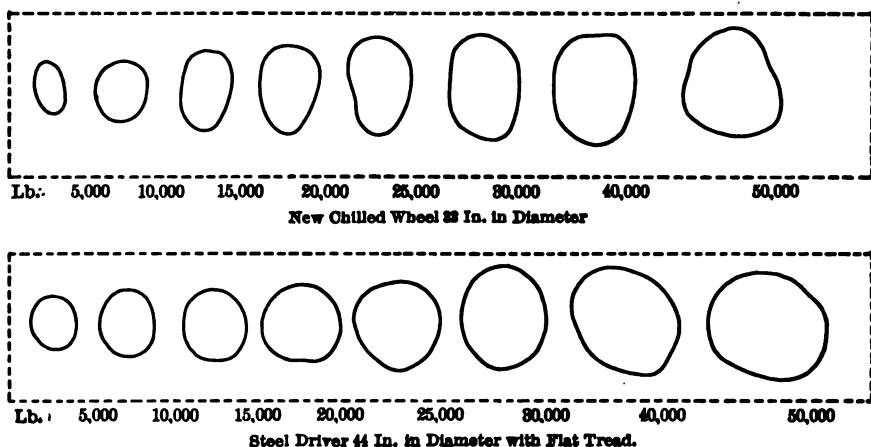


FIG. 45.—Wheel Impressions on a 75-lb. Rail. Top Radius, 14 in. Areas are One-half Size.

it is evident that the metal near the center of the compressed surface is under greater compression and greater lateral restraint due to the wider ring of metal and will have a higher elastic limit than in the second case.

In Fig. 45 are shown a series of actual areas of contact obtained by pressing sections of a cast-iron car-wheel and of a locomotive steel driving-wheel upon the cylindrical top surface of a steel rail. This was done in a testing-machine in such a way that there was no rocking motion and the area of contact was clearly distinguished.\*

The areas of these surfaces of contact were determined by a planimeter, and these are plotted to their corresponding loads in Fig. 46. It will be seen that these plot in nearly a straight line through the origin. If such a law be assumed, it follows:

1. That the area of contact increases directly with the load.
2. That the mean intensity of pressure is a constant for all loads.

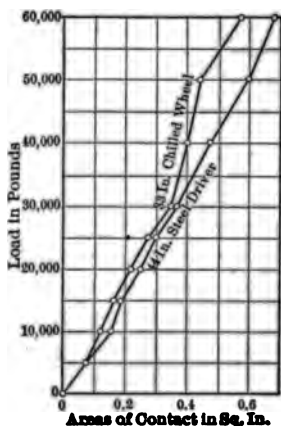


FIG. 46.—Showing Relation between Total Load and Area of Contact between Wheels and Rails. (Johnson, *Trans. A. S. C. E.*, Vol. 32.)

\* See a full account of these tests, showing other areas of contact, in *Trans. Am. Soc. Civ. Engr.*, Vol. 32, p. 270, 1894.

3. That in these experiments this mean intensity of compressive stress, for all loads, was about 82,000 lb. per square inch.

4. Since the maximum deformation (at the centers of these areas) is twice the average deformation (assuming the volumetric deformation to be that of a segment of a paraboloid of revolution) then the maximum compressive-stress intensity for all loads is about 164,000 lb. per square inch.

5. Since no measurable permanent set was produced by any of these loads on either wheels or rail, it follows that the "apparent elastic limits" of the materials had not been reached for this condition of contact, although the ordinary elastic limit of the rail material, for a free flow as in Fig. 45a was about 50,000 lb. per square inch.

These are important conclusions, and should be supplemented and supported by further observations of this character.

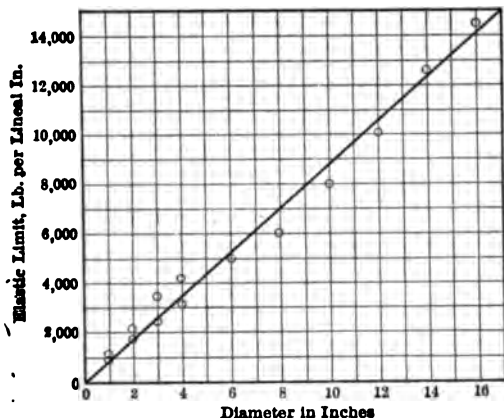


Fig. 47.—The Elastic Limit per Lineal Inch of Rollers of Various Diameters. (Crandall and Marston, in *Trans. Am. Soc. Civ. Engr.*, Vol. 32, p. 120.)

In Fig. 47 are shown the results of tests made by Prof. Crandall and Marston to find the elastic-limit loads on steel cylinders resting on or between steel plates. These

results show that the elastic loads vary directly with the diameters, these loads per lineal inch of rollers, for mild structural steel, being

$$p = 880d, \quad \dots \dots \dots (1)$$

where  $p$  = elastic-limit load in pounds per lineal inch, and  $d$  = diameter of roller in inches.

**724. Properties of Wire.**—We have already considered the effects of drawing, patenting and annealing on the properties of wire in Art. 716. To determine the quality of wire it has been customary to subject specimens to tensile, cold bending and torsion tests. If the tension test is so made that a complete stress-deformation diagram is obtained, it appears to furnish the essential information. From such a test the strength, elastic limit, and ductility can be read directly, and the toughness can be roughly estimated from the area of the diagram. The per cent elongation is not only an index of ductility but is also a valuable criterion of the wearing quality of the wire when made into rope. The cold-bend test is usually made by clamping one end of the

wire between jaws having a radius of  $\frac{1}{8}$  in., or equal to the diameter of the wire, and bending the projecting portion back and forth through an angle of  $180^\circ$  until failure occurs. The number of bends required for rupture constitutes a measure of the capacity of the wire to receive cold work and also indicates its ductility. Torsion tests are usually made on a gauge length of 8 in., and the number of turns which the specimen can withstand under a constant tension is determined. With autographic machines, like Fig. 16, Ch. II, the stress-diagram in torsion is gotten. The torsion test appears to be of doubtful value except as a measure of uniformity in the material.

The strength and ductility of wire depend upon the drawing and heat treatment and also upon the diameter of the finished wire. Annealed iron wire has a tensile strength of 50,000 to 60,000 lb. per square inch. The same material hard drawn with a diameter of an eighth of an inch will have a strength of 70,000 to 80,000 lb. per square inch, when drawn to a very fine wire it may have a strength of a 100,000 lb. per square inch. The strength of steel wire is of even greater range, depending upon composition in addition to the previously mentioned factors. Thus we find low-carbon hard-drawn wire with an ultimate strength of 90,000 to 100,000 lb. per square inch and high-carbon hard-drawn piano wire with a strength of 300,000 to 400,000 lb. per square inch. A strength of 447,000 lb. per square inch is recorded for a piano wire 0.03 in. in diameter containing 0.80 per cent carbon.\*

Mr. J. Bucknall Smith gives the following average values of strength for wire used in wire rope: †

Kind of Wire.	Ultimate Strength (lb./in. <sup>2</sup> )
Bright hard-drawn iron wire . . . . .	80,000
Bessemer steel wire . . . . .	90,000
Mild open-hearth steel wire . . . . .	130,000
High-carbon open-hearth steel wire . . . . .	180,000
Crucible cast-steel wire (patented) . . . . .	220,000
Crucible cast-steel wire (plow quality) . . . . .	240,000

"Bright wire" is that which remains untreated after the final drawing; if annealed or tempered it is left black. Plow-steel wire is so called because it was first used for drawing machine-plows in England.

The proportional limit of hard-drawn wire is from 65 to 80 per cent of its ultimate strength.

**725. Wire Rope.**—Common wire rope is twisted from six strands, each of which consists of 7 to 19 individual wires. In ordinary rope the strands are twisted in the opposite direction to the twist of the wire in the strands; but in rope which must have high resistance to abrasion both wires and strands are twisted in the same direction, thus forming

\* *Tests of Metals*, 1894, p. 347.

† *Mining Journal*, June 6-July 11, 1896.



the Lang-lay rope. Where great flexibility is desired the rope is provided with a hemp core, or, if the wear is not great, a larger number of wires may be used in the strands. For elevators and other hoisting purposes where the loads are light, the speeds high and the wear on sheaves is considerable, an iron wire rope gives good service. For derricks, mine hoists, cableways, conveyors and uses where considerable strength and abrasive resistance is required, cast-steel wire rope has the requisite strength and durability. Plow-steel wire rope is used in deep mine hoists, on dredges, stump pullers, and under conditions where maximum strength and toughness are wanted. For the fixed lines in aerial cableways where very great wearing resistance is wanted, a rope with a smoother surface is sometimes used. One of the two common types is the steel-clad wire rope in which each strand is spirally wound

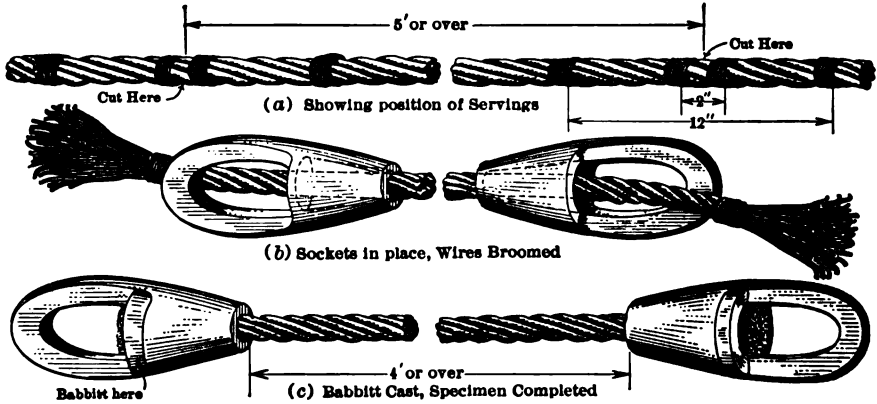


FIG. 48.—Method of Socketing Wire Rope.

with flat steel strips. The other is the locked-wire rope in which the surface layer is made of wires drawn to such a shape that they interlock when twisted about the rope and form a true cylindrical surface. Flat ropes are also used for hoisting purposes.

The strength of wire rope is difficult to obtain from short specimens because of the small stretch of the wires and the non-uniformity in stretch due to variation in the rigidity with which they are held. A fairly satisfactory method of gripping the ends follows. A length of 5 ft. or more should be marked off on the rope and two sets of four servings (or windings) with black iron wire should be made at each end of the marked length as shown in Fig. 48a. The rope should be cut at the marks by means of a cold chisel and the servings will prevent unraveling. Conical sockets may now be slipped over the ends of the rope, the outer servings removed and the wires broomed as shown in Fig. 48b. If there is a hemp core it should be cut back to the inner servings. The wires are cleansed by dipping in gasoline

followed by hot caustic potash. After cleaning, the wires should be dipped in zinc chloride and thoroughly tinned in the molten babbitt used to fill the socket. The sockets can then be pulled over the cable ends and the babbitt poured into the socket in such manner as to completely fill it. Alloys used with success at the University of Wisconsin are: Lead, 83 per cent; tin, 7 per cent; antimony 10 per cent, also lead, 60 per cent; tin, 30 per cent; antimony, 9 per cent, and bismuth, 1 per cent.\* Both of these melt below 550° F. The second alloy was previously used with good results at the Watertown Arsenal. After socketing, the specimen is ready for the testing machine.

In Table 11 is given a summary of several hundred tests on high-grade wire and on ropes made of it. These tests were conducted with great care by Tetmajer. The tension and torsion results on wires were

TABLE 11.—RÉSUMÉ OF TESTS ON CRUCIBLE CAST-STEEL WIRE AND WIRE ROPE USED ON CABLE RAILWAYS IN SWITZERLAND.

(From *Tetmajer's Communications*, Vol. 4, p. 272.)

Number of Cable.	TEST OF ENTIRE CABLE. (EACH THE MEAN OF TWO TESTS.)			TEST OF INDIVIDUAL WIRES. (EACH THE MEAN OF ELEVEN TESTS.)					Ratio of Strength of Cable to Average Strength of Wires.
	Diameter in Inches.	Tensile Strength in Pounds per Square Inch of Actual Wire Section.	Per Cent of Elongation in 8 ft.	Tension Test.			Torsion	Bending.	
				Strength in Pounds per Square Inch.	Per cent of Elongation in 20 Inches at Rupture.	Work of Deformation in Foot-pounds per Cubic Inch.	Number of Twists of (360°) in 8 Inches.	Number of Bends of 180° Each on $\frac{1}{4}$ -in. Radius.	
1	1.65	220,000	3.12	265,000	3.4	6,400	27.5	11.4	83
2	1.67	117,000	7.45	122,000	9.4	9,500	61.5	11.8	96
3	1.18	205,000	2.61	213,000	3.0	4,600	35.1	17.8	96
4	1.43	191,000	3.30	207,000	3.4	5,000	44.5	18.0	93
5	1.00	184,000	3.92	191,000	3.85	5,300	.....	15.1	96
6	1.00	184,000	3.28	190,000	4.0	5,700	52.6	14.8	97
7	1.38	180,000	2.37	222,000	3.0	4,600	33.7	11.0	77
8	1.30	226,000	3.00	247,000	3.3	5,700	21.7	9.6	92
9	1.26	210,000	3.15	238,000	3.3	5,400	31.1	9.4	89
10	1.00	190,000	2.40	190,000	2.7	3,400	48.1	18.8	100
Average (omitting No. 2).		199,000	2.98	217,000	3.29	5,100	36.8	14.0	92

determined from autographic diagrams, consequently the ductility measurements include the elastic deformations. For a wire having a strength

\* Tetmajer used an alloy of 8 parts tin, 1 part antimony and 1 part copper for iron and mild steel wires; for hard steel wires he used 9 parts lead, 2 parts antimony and 1 part bismuth.

of 200,000 lb. per square inch the elastic stretch amounts to 0.7 per cent. Consequently the per cent elongation as measured across the break after rupture was about 0.7 or 0.8 per cent less than the values recorded in the sixth column of the table.

In long wire ropes on a straight pull the strength of the rope may be taken as about equal to the average strength of the individual wires if these are all of about the same ductility and ultimate strength. If the wires differ greatly in ductility, the ultimate strength of the rope is the average resistance of the wires at that percentage of elongation which corresponds to the total elongation of the least ductile samples. It is common to assume the rope to have 85 per cent of the total strength of the wires when tested individually. The total area ( $a$ ) of the wires in a rope may be calculated from the following: Ordinary rope with hemp core,  $a = .46d^2$ ; ordinary rope with steel core,  $a = .50d^2$ ; locked wire rope,  $a = .74d^2$ ; where  $d$  = diameter of rope. The weights in pounds per lineal foot ( $w$ ) of the same ropes may be estimated by the following equations:  $w = 1.58d^2$ ,  $w = 1.70d^2$ , and  $w = 2.5d^2$ , respectively. A factor of safety of 5 is commonly used in figuring working loads, but for passenger elevators and conditions where failure in the rope may endanger life, the factor should be increased to 8. The modulus of elasticity of ordinary wire rope varies between 7,000,000 and 10,000,000 lb. per square inch of metal in the cross-section.

Wire-rope pulleys, sheaves, and barrels should have a diameter not less than thirty times the circumference (or say one hundred times the diameter) of the ropes running upon them, to prevent excessive bending strains in the ropes.

### STEEL UNDER COMBINED STRESS

**726. Effects of Combined Stress upon the Elastic Limit.**—A statement concerning the theories underlying the causes of elastic breakdown has been made in Art. 41. We shall now briefly consider some experimental evidence showing the weakening effects produced by a combination of torsion and direct or bending stresses.

The influence of combined stress in lowering the elastic limit is shown in Fig. 49. These results were obtained by Hancock at Purdue University \* from experiments on steel tubing and steel shafting. The deformations both in tension and in torsion were measured with apparatus of sufficient accuracy to permit the determination of the elastic limit (really the limit of proportionality). It seems rather doubtful if the measurements of deflection were sufficiently accurate to detect the overstrain in the extreme fibers of the specimens under flexure.

\* Reported in *Proc. A. S. T. M.*, Vol. 7, p. 267.

Curves 1, 2 and 3, represent the effect of an initial torsional moment in lowering the elastic limit in flexure, compression and tension, respectively. Thus, if an initial twisting moment sufficient to stress the outside fiber of a round bar to one-half of its elastic limit in shear is applied and the bar is then axially compressed under progressively increasing loads, curve 2 shows that the elastic limit in compression will be found at 79 per cent of the load which the bar could carry under simple compression. It appears that curve 1 is too high, probably

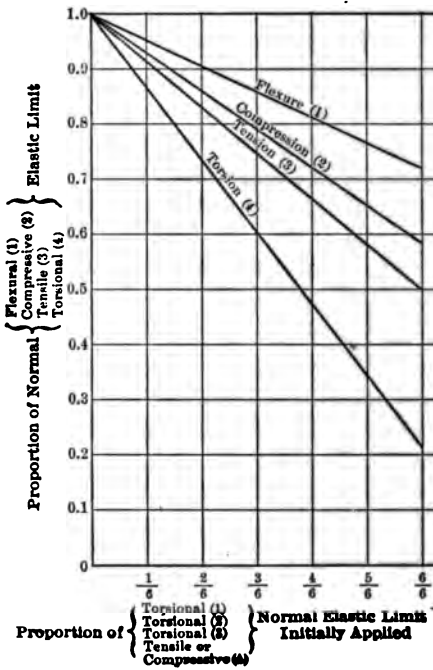


FIG. 49.

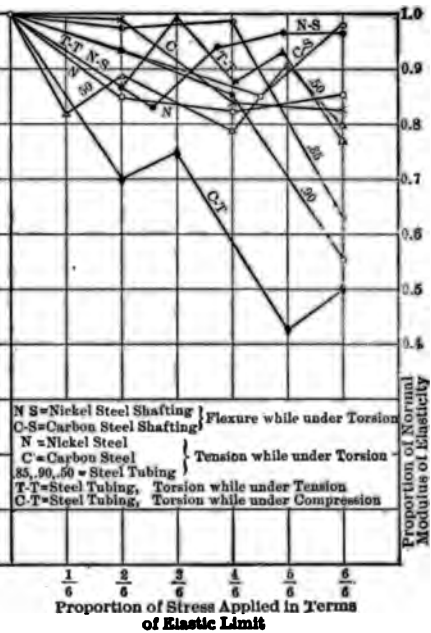


FIG. 50.

FIG. 49.—The Influence of Combined Stress on the Elastic Limit. (Hancock.)

FIG. 50.—The Influence of Combined Stress on the Modulus of Elasticity. (Hancock.)

due to the difficulty of determining by deflection increments when the outside fiber had been overstrained. Curve 4 shows the influence of an initial tensile or compressive stress on a round bar subjected to torsion.

#### 727. Effect of Combined Stress on the Modulus of Elasticity.—

Hancock's tests also show that the modulus of elasticity in tension and in torsion is considerably decreased by combined stress, whereas the effect of combined stress on the modulus of elasticity in flexure is less, Fig. 50. The results are, however, much more irregular than those from which Fig. 49 was derived.

## CHAPTER XXIV

### ALLOY STEELS \*

**728. Varieties of Alloy Steels and Their Manufacture.**—Those steels which owe their peculiar properties to the presence of one or more elements besides carbon, or to the joint action of such elements and carbon, are termed alloy steels. In a general way we may say that these alloying elements influence the properties of steels through the changes which they affect, (1) in constitution and structure, (2) in shifting the position of the critical ranges on heating and cooling, and (3) in promoting stability of structure under wide temperature variations. When the distinctive properties are produced by the addition of one element to a carbon steel, a *ternary alloy*, consisting of iron, carbon and the element, is formed. When two such elements are added to a carbon steel a *quaternary alloy* results. The more important alloy steels are nickel steel, manganese steel, chrome steel, tungsten steel, vanadium steel, silicon steel—all of which are ternary alloys—the quaternary alloys, chrome-nickel and chrome-vanadium steels, and the high-speed steels which are more complex alloys.

The alloy steels which are turned out in large tonnages like the nickel, silicon, chrome-nickel, chrome-vanadium and manganese steels are generally manufactured in the acid open-hearth furnace. Nickel may be added with the charge, but the other elements are added just before pouring, or in the ladle, to prevent losses through oxidation. Tungsten steels and high-speed steels are generally made in the crucible or electric furnace.

**792. Nickel Steel** is, from the standpoint of tonnage and variety of uses, the most important of the ternary alloys. Although additions of nickel to pure iron † effect large increases in strength and hardness with corresponding losses in ductility, the commercial alloys generally contain carbon with small percentages of the other impurities common to carbon steels. Nickel apparently forms a solid solution with iron, reduces the solubility of carbon in iron, and produces marked changes in the position

\* References: *Steel and Its Heat Treatment*, by D. K. Bullens; "Topical Discussion on the Role of the Several Alloying Elements in Alloy Steels," *Proc. A. S. T. M.*, Vol. 17, p. 5-57; *Bull. No. 100 U. S. Bureau of Mines; Manufacture and Uses of Alloy Steel*, by H. D. Hibbard; *The Physico-Chemical Properties of Steel*, by C. A. Edwards.

† *Bull. No. 346, Univ. of Wisconsin*, by Burgess and Aston.

of the critical range. On account of these influences the constitution, the mechanical and the electrical properties of nickel steel are greatly altered by variations in the nickel content.

The relation of nickel and carbon content to the constitution of nickel steel as cast is shown in Fig. 1a. Fig. 1b shows the influence of nickel on the magnetic point ( $A_{r2}$  or  $A_{c2}$ ) or nickel-iron alloys. It will be noted that alloys containing less than 25 per cent nickel are irreversible (transformations in heating take place at much higher temperatures than in cooling), but alloys containing over 25 per cent of nickel are reversible. Although evidence is incomplete, the critical ranges of nickel steels are

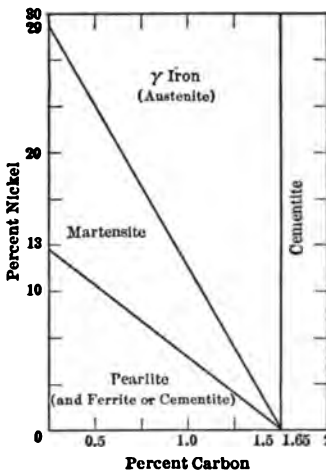


FIG. 1a.

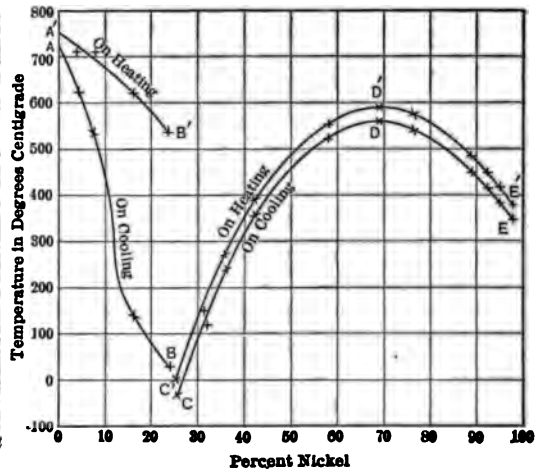


FIG. 1b.

FIG. 1a.—The Constitution of Nickel Steels. (Guillet, *Jour. Ir. and St. Inst.*, Vol. 70, p. 4.)

FIG. 1b.—Effect of Nickel on Critical Points. (Osmond, *Comptes Rendus*, Vol. 123, p. 306.)

effected in somewhat the same manner as the above. Since nickel lowers the solubility of carbon in iron, it is not possible to carry over .50 to .60 per cent carbon in nickel steels which are to be annealed on account of the tendency of carbon to separate on slow cooling from above the critical range and form graphite.

From a consideration of the constitution diagram (Fig. 1a) we should expect the low-nickel (pearlitic) steels to possess high strength and good ductility, those of medium-nickel content (martensitic) we should expect to be much stronger and far more brittle while those of high-nickel content (austenitic) should be strong and ductile. These expectations are roughly borne out by the experimental results of Giesen which are plotted in Fig. 2. In toughness the austenitic nickel steels of a given carbon content

are superior to the pearlitic, which in turn greatly surpass the martensitic steels. Because of these mechanical properties, the low and high-nickel steels are of most use.

**Low-nickel Steels.**—On account of the high cost of nickel only the low-nickel steels have entered the structural field. Most of the structural nickel steels contain 3 to 4.5 per cent of nickel and 0.15 to 0.40 per cent carbon. Advocates of structural nickel steel claim that it has greater toughness and ductility than a carbon steel of equal strength. It is also claimed that it is free from segregation but is more liable to blow-holes

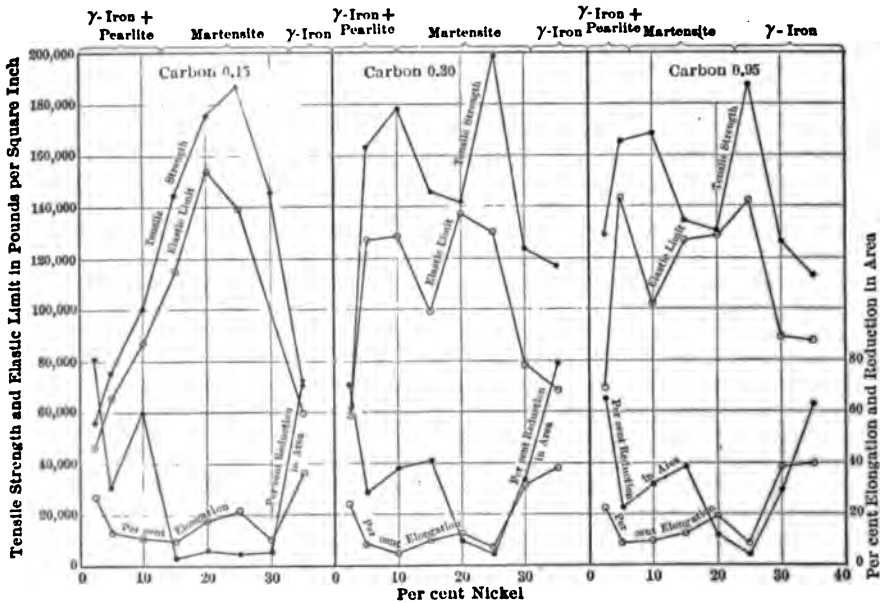


FIG. 2.—The Influence of Nickel on the Strength and Ductility of Steel. (Giesen, *Carnegie Fellowship Memoirs*, Vol. 1, p. 1.)

and fissures than carbon steel. It possesses excellent machining qualities but cannot be welded. Low-nickel steels are somewhat superior to carbon steels in resisting corrosion; high-nickel steels are very markedly superior.

Abbott \* states that each per cent of nickel added to the low-nickel steels ( $Ni < 8$  per cent) lowers  $A_{c1}$  about  $10.3\text{ }^{\circ}\text{C}^{\circ}$ . The other critical points are also depressed nearly as much. On cooling, the lowering effect on the critical points of these steels is nearly twice as great as in heating.

When heat-treating nickel steels it is necessary to hold them longer than carbon steels at temperatures above the upper critical point in order that transformations may be completed. Since nickel steels are less liable

\* *Proc. A. S. T. M.*, Vol. 17, p. 13.

to injury through overheating than carbon steels, double quenching affords an excellent although somewhat costly method of equalizing the structure in hardened steel. Double quenching consists in heating the steel to a temperature considerably in excess of the upper critical point, quenching, reheating to a temperature just above the upper  $A_c$  point to refine the grain and again quenching. Otherwise the heat treatment of low-nickel steels is carried on in manner similar to treatment of carbon steels.

Abbott also states that the strength of a given carbon steel is increased by 4200 lb. per sq. in. for each per cent of nickel added up to 8 per cent and the elongation is reduced by 1 per cent. Thus a 0.30 per cent carbon steel having an ultimate tensile strength of 70,000 lb. per sq. in. would have its strength increased to 85,000 lb. per sq. in. by the addition of 3.5 per cent of nickel and its elongation reduced 3.5 per cent. With  $\frac{3}{4}$ -in. bars he states that the tensile strength of hardened and drawn nickel steel is increased by (29400–21.4 T) lb. per sq. in. for each per cent increase in nickel, T indicating the drawing temperature in degrees C. The elastic limit is likewise raised by (28700–17.4 T) lb. per sq. in. for each per cent of nickel added. Hardening lowers the per cent elongation about 0.73 per cent for each per cent of nickel and drawing does not affect this reduction appreciably. Comparing with carbon steels, we note that structural nickel steels have greater resistance to impact than carbon steels \* for like uses. Heat treatment seems to make the superiority of the nickel steels more marked.

These steels are used considerably in long-span bridge construction, for shafting, rifle barrels, ordnance, axles, bearings, forgings, and castings.

*High-nickel steels* containing from 30 to 35 per cent of nickel and less than 0.40 per cent carbon possess great resistance to shock, a tensile strength of 85,000 to 95,000 lb. per sq. in., an elastic limit of half that amount, and an elongation of 35 to 40 per cent in 2 in. The modulus of elasticity of these steels is low, being about 23,000,000 lb. per sq. in. Since these high-nickel steels have very good resistance to corrosion they are well adapted to valves and spindles for gas engines, boiler tubes, and for wire netting and cables which are to be used in salt water.

Steels with 25 per cent of nickel are practically impermeable to magnetism unless cooled below  $-50^{\circ}\text{C}$ . when they become magnetic and remain so after returning to atmospheric temperatures (see Fig. 1b.) Steels of 30 to 35 per cent nickel are magnetic at ordinary temperatures but have much lower permeability than carbon steels. They do not suffer material change in permeability due to heat treatment.

*Invar*, an alloy of iron with 36 per cent nickel and 0.50 per cent carbon, has a coefficient of expansion of only 0.000001 per  $^{\circ}\text{C}$ . It is most useful in making steel measuring tapes and bars, clock pendulums and

\* See Tests by W. K. Hatt in *Trans. Am. Soc. C. E.*, Vol. 63, p. 307.



other apparatus in which dimensional changes due to temperature must be made as small as possible. *Platinite*, a nickel steel containing about 46 per cent nickel and a low percentage of carbon, finds useful application because it has the same coefficient of thermal expansion as glass.

**730. Manganese steel** is the term applied commercially to alloys of iron containing from 10 to 14 per cent of manganese and 1 to 1.3 per cent of carbon. Inasmuch as manganese exerts twice as powerful an influence as nickel in lowering the critical temperatures of the iron-carbon alloys and also renders the rate of transformation at the critical points

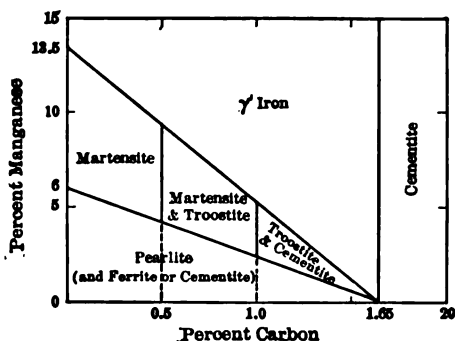


Fig. 3.—The Constitution of Manganese Steels (Guillet, *Jour. Ir. and St. Inst.*, Vol. 70, p. 7.)

very sluggish, it exercises a most potent effect on the constitution and properties of the metal. Fig. 3 shows the variations in structures of the alloys due to changes in manganese and carbon content. It will be noted that the commercial manganese steels are austenitic and experiments have shown that they have no well-defined critical points. In cast or rolled form, the steel is glassy brittle owing to the separation of manganese-iron carbide which, in slow cooling, forms

a weakening network around the austenite grains. By reheating the steel to a temperature between 1000 and 1100° C. and quenching in water, separation of the carbide is prevented and a fine-grained uniform austenitic structure is secured. As a consequence of this heat treatment the ductility and toughness is wonderfully improved.

Fig. 4 shows the tensile properties of various manganese steels as determined by Hadfield, who first investigated their properties. It will be observed that the ultimate strength of the water-toughened specimens of the commercial alloys is about 140,000 lb. per sq. in. and the elongation between 40 and 50 per cent. The elastic limit is, however, quite low, being only 50,000 lb. per sq. in. The steel exhibits no necking in the tensile test and consequently has a low per cent reduction in area. In shock resistance, it is far superior to carbon steels of equal strength. The Brinell hardness number of the commercial steel is only 200, or thereabouts, but it possesses extremely high resistance to certain types of abrasion. Howe \* attributes this resistance to its capacity of hardening under deformation. The austenitic manganese steels are non-magnetic and possess high electrical resistance.

On account of the impossibility of softening by heat treatment, man-

\* *Proc. A. S. T. M.*, Vol. 17, p. 7.

ganese steel cannot be machined. Therefore it can only be used advantageously in shapes which can be cast or rolled to approximately final form and finished by a comparatively small amount of grinding.

Water-toughened manganese steel is effectively utilized for rails on curves where wear is severe, for frogs and switches, for vaults and safes, for screens in separating stone, for crusher jaws and rolls.

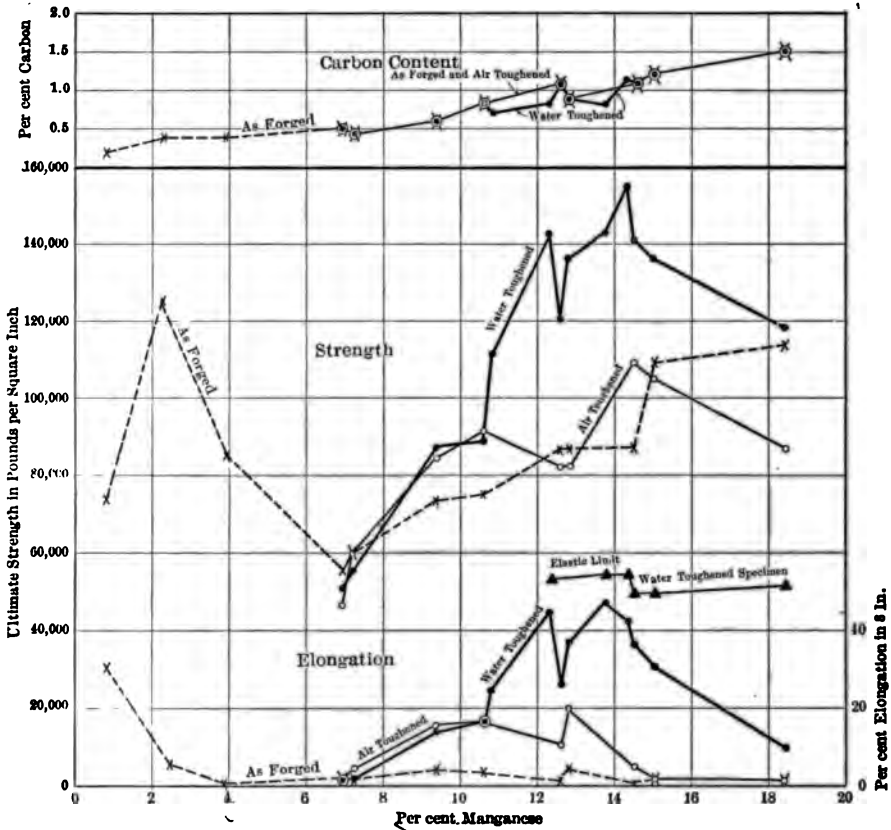


FIG. 4. -The Influence of Manganese on the Strength and Ductility of Steel. Specimens were  $\frac{3}{4}$  in. in diameter. (Hadfield, *Proc. Inst. Civ. Engr.*, Vol. 93, p. 1.)

The martensitic manganese steels are of little commercial value on account of their excessive brittleness.

**731.** Chrome steels containing 0.5 to 2 per cent chromium and 0.2 to 1.5 per cent carbon find considerable application for parts where great hardness, high strength and a fair degree of toughness are required. The great hardening capacity of the chrome steels is due to the combination of chromium with cementite, thus forming a double carbide with iron. Chromium also promotes a very fine-grained structure in the metal.

Steels of the composition mentioned above are pearlitic in the normal state. In the annealed state they have somewhat greater ductility than carbon steels of equal strength, but subsequent to proper quenching and tempering, they are much stronger and harder, and somewhat tougher and more ductile than carbon steels. Chrome steels also have a very high elastic ratio. In toughness, however, they are excelled by the chrome-nickel and chrome-vanadium steels.

Since austenite does not readily dissolve the double carbide it is necessary in annealing chrome steels to soak them for several days at a temperature just above the critical range, which is 25 to 50° C. higher than for carbon steels of like carbon content, and then cool very slowly. Excepting the differences in critical points, hardening and drawing of chrome steels are accomplished in much the same way as with carbon steels. Table I shows the influence of hardening and tempering on the properties of several varieties of chrome steel.

TABLE 1.—MECHANICAL PROPERTIES OF CHROME STEELS

Chemical Analysis.	Treatment.		Strength in Pounds Per Square Inch.		Percent Elongation in 2 Inches.	Brinell Hardness.	Authority.
	Quenched at, ° C.	Drawn at, ° C.	Ultimate.	Elastic Limit.			
C=0.64 Cr=1.04 Mn=0.28 Si=0.17	870	400	227,500	170,000	5.0	477	* Bullens
	870	500	212,000	155,000	8.0	444	
	870	600	186,000	127,500	10.0	387	
	As forged		78,800	51,100	30.5	AC <sub>3</sub>	
C=0.20 Cr=1.98 Mn=0.12	800	400	153,600	150,100	12.5	822°C.	† McWilliam
	800	550	130,000	123,200	16.0		
	800	700	92,300	71,600	28.0		
	Annealed		73,900	35,800	40.5		
	As forged		120,500	71,700	20.5	778°C.	and
C=0.50 Cr=1.99 Mn=0.24	800	400	255,700	251,000	9.0		
	800	550	201,000	190,500	13.0		
	800	700	139,800	127,700	21.0		
	Annealed		84,200	28,700	28.0	777°C.	Barnes
	As forged		170,000	116,500	10.0		
C=0.85 Cr=2.00 Mn=0.24	800	400	.....	.....	.....		
	800	550	214,200	207,000	8.5		
	800	700	141,100	128,800	20.0		
	Annealed		89,800	42,100	32.0		

\* Specimens used by Bullens (see *Steel and Its Heat Treatment*, p. 302) were 1 inch rounds quenched in oil.

† Specimens of McWilliam and Barnes (see *Jour. Iron and Steel Inst.*, Vol. 81, p. 253) were quenched in water. They were 0.564 inch in diameter.

Like nickel steels, the chrome steels are also highly resistant to corrosion.

Steels with about 0.5 per cent of chromium and 0.60 to 0.90 per cent carbon are used for chipping chisels, drills, razors and saw-blades. Files are made from 0.5 per cent chrome steel containing about 1.5 per cent carbon. Steels containing about 1.0 to 1.5 per cent chromium and 0.90 to 1.10 per cent carbon are used in balls and rollers for bearings, also for safes and crusher jaws. Armor-piercing projectiles, rolls for cold-rolling metals, and drawing dies are made from steels containing about 2 per cent chromium.

**732.** Tungsten steel is the oldest of the alloy steels but is of minor importance at the present time. The chief use now made of this ternary alloy is in permanent magnets, for which purpose the steel is unexcelled. Magnet steels usually contain about 0.6 per cent of carbon and 6 per cent of tungsten. After the magnets have been formed they are hardened and then soaked for a long time at the temperature of boiling water in order to stabilize their magnetic properties. Steels with about 3 per cent of tungsten and 1 per cent of carbon are used to some extent for lathe tools for rapidly finishing iron and steel parts. Such steels are best treated in much the same manner as carbon steels.

Tungsten steels are very complex in constitution. According to results which Arnold and Read \* obtained by experimenting on steels containing 7.0 per cent carbon, if in annealed steels the ratio of carbon to tungsten is 1 : 15.3 these elements are united as a carbide (WC); if the carbon is in excess of this ratio both carbides of iron and tungsten are present; if tungsten is in excess there is no carbide of iron, but a mixture of carbide of tungsten and tungstide of iron ( $\text{Fe}_2\text{W}$ ) is present. When the tungsten steels are heated to temperatures above  $1000^\circ\text{C}$ . the lower critical point is depressed very markedly. This effect of tungsten on the critical point  $A_1$  has a very important bearing on the properties of the high-speed steels of which tungsten is a part.

Swinden's researches † show that the strength of 3 per cent tungsten steels increases from 74,000 to 139,000 lb. per sq. in. as the carbon content is raised from 0.2 to 0.9 per cent, the per cent elongation in 2 in. falling from 28 to 13.5 per cent for the same change in composition. Further increase in carbon produced a decrease in strength. With steels containing 0.8 per cent carbon and tungsten ranging up to 19 per cent Portevin ‡ obtained a maximum strength of 192,000 lb. per sq. in. with a tungsten content of 10 per cent. The steel was, however, quite brittle.

\* *Proc. Inst. Mech. Engr.*, 1914, Pt. 2, p. 223.

† *Jour. Iron and Steel Inst.*, 1907, Pt. 1, p. 294.

‡ *Carnegie Fellowship Mem.*, Vol. 1, p. 261.

Both the strength and hardness of tungsten steels can be raised materially by heat treatment.

**733. Vanadium Steel.**—Although the strength and hardness of steel is improved by additions of vanadium up to 1 per cent, the element is so expensive that the content usually is kept much lower, running from 0.10 to 0.20 per cent in commercial vanadium steels. Vanadium is one of the most powerful of the alloying elements in raising strength and hardness. Vanadium is also a powerful scavenger of oxygen but is too costly to use for such purposes.

In low percentages, vanadium combines with both the ferrite and pearlite of the steel. In the latter it enters the cementite as a double carbide with iron and thus exerts a pronounced hardening influence. Vanadium exerts a more powerful effect than chromium in raising the upper critical points ( $A_2$  and  $A_3$ ). Consequently vanadium steels require higher temperatures (30 to 40° C.) for hardening and annealing than carbon steels. Vanadium renders the coalescence of the pearlite and cementite much more sluggish and, therefore, promotes the formation of globular or sorbitic pearlite in the normally cooled steels. On account of this tendency, also common to the chrome steels, vanadium steels are much tougher than carbon steels of like strength. Vanadium also raises the temperature to which a steel may be heated without material loss of strength and hardness (Art. 815–819). This latter property renders vanadium a useful alloying element in high-speed steels.

Experiments by McWilliam and Barnes\* on 0.2 per cent vanadium steels show that the tensile strength of untreated steels varied directly from approximately 85,000 to 157,000 lb. per sq. in. as the carbon content increased from 0.09 to 0.71 per cent. The elongation in 2 in. was reduced from 23 per cent with 0.09 per cent carbon steels to 9.5 per cent with 0.98 per cent carbon steels. When quenched from 850° C. and tempered at 400° C. the strength increased from 61,000 to 208,000 lb. per sq. in. as the carbon content was raised from 0.09 to 0.98 per cent. The ductility of the treated steels was reduced from 33 to 11 per cent for the same increase in carbon. The yield point of these steels ranged between 75 and 85 per cent of the ultimate strength. Carbon beyond 1 per cent does not appear to benefit the strength of vanadium steels.

Vanadium steel is used to some extent in making castings for locomotive frames, for forgings, automobile axles, springs and battering tools. The ternary vanadium steels have been displaced very largely, however, by the superior chrome-vanadium steels.

**734. Silicon Steels.**—Owing to difficulty in rolling, steels containing more than 5 per cent silicon are not of commercial value. The influence of silicon on the mechanical properties of steel is similar to that of carbon

\* *Jour. Iron and Steel Inst.*, Vol. 83, p. 294.

but is proportionately much less powerful. Hadfield \* found that the effect of silicon in increasing the strength of unannealed steel rose from 8000 lb. per sq. in. for  $1\frac{1}{2}$  per cent silicon to 27,000 lb. per sq. in. for  $3\frac{1}{2}$  per cent silicon. In his experiments the elastic ratio varied between .73 and .80 and the elongation in 2 in. decreased from 31 to 14 per cent with increase of silicon from  $1\frac{1}{2}$  to  $3\frac{1}{2}$  per cent. By alloying silicon with pure iron, Yensen † found that the strength was increased from 38,000 to approximately 85,000 lb. per sq. in. as the silicon content was increased from 0 to 4 per cent.

Silicon steel is used to some extent for automobile gears and springs. For such purposes it contains carbon 0.40 to 0.50, manganese 0.50 to 0.70 and silicon 1.25 to 2.00 per cent. These steels are often termed silico-manganese steels, although there appears little justification for the term, since the manganese content is not abnormally high. After suitable quenching and drawing treatment silico-manganese steels have tensile strengths of 200,000 lb. per sq. in., or more, with an elongation in 2 in. of 8 to 10 per cent. Silico-manganese steels are less expensive than the chrome-nickel or chrome-vanadium alloys but do not appear to be as tough and, according to Bullens, are quite difficult to heat treat successfully.

On account of its cheapness, low hysteresis loss, and high permeability to magnetism, steel containing about 4 per cent of silicon and very little carbon or manganese is the best material known for pole pieces of dynamos and for transformer cores.

**735. Chrome-nickel steels** constitute one of the most important classes of alloy steels. The properties rendering these steels of high commercial value are very high strength, elastic limit and hardness combined with good ductility, and a high degree of toughness, all of which may be secured by proper heat treatment.

The most valuable steels of this class are those which are normally pearlitic, those containing pearlite and ferrite ( $C < 0.85$  per cent) being used much more than steels containing pearlite and carbide ( $C > 0.85$  per cent). In the commercial chrome-nickel steels the composition ranges are about as follows:

Type	Cr	Ni	C	Mn	Si	P	S
Low . . . . .	0.5-0.7	1.0-1.5	.20-.40	.50-.80	Low	<.04	<.04
Medium . . .	0.9-1.2	1.5-2.0	.20-.50	.30-.60	Low	<.04	<.04
High . . . . .	0.7-1.8	2.5-3.5	.20-.50	.50-.70	Low	<.04	<.04

\* *Jour. Iron and Steel Inst.*, Vol. 2, p. 212.

† *Bull. No. 83, Engr. Expt. Sta., Univ. of Illinois.*

Bullens \* says that the most effective ratio of nickel to chromium is approximately  $2\frac{1}{2}$  to 1.

In steels of the above composition it appears that the influences of nickel and chromium on strength are cumulative. In comparing with nickel steels of like strength the quaternary alloys have greater hardness and higher elastic limit due to the presence of chromium; furthermore they are somewhat less costly. Again comparing with the chrome steels of equal strength, it appears that the chrome-nickel steels are less hard and have a somewhat lower elastic ratio but are much tougher, more ductile and less liable to injury through overheating than the former.

Bullens states that the critical range of these steels is lower than for carbon steels and suggests the following as a suitable method for heat treating gear stock of chrome-nickel steel:

- " (1) Quench in oil from 175 to 200° F. over the critical range.
- " (2) Quench in oil from about 50° F. over the critical range.
- " (3) Anneal at about 75° F. under the critical range.
- " (4) Machine.
- " (5) Quench in proper medium from about 50° F. over the range.
- " (6) Draw the temper to suit the work in hand."

For shafting and axles requiring a toughening treatment the above set of operations may be abridged by following operations (1) and (2) with a drawing treatment at 900° F.

The effects of drawing temperature on the mechanical properties of a low-carbon low-chrome-nickel and a medium-carbon low-chrome-nickel steel are shown in Fig. 5. The strength and ductility of steels containing larger percentages of chromium and nickel are practically the same as those of low-chrome-nickel provided the carbon content is constant. Bullens claims, however, that the dynamic and endurance properties of the high-chrome-nickel steels after heat treatment are materially superior to those of the low-chrome-nickel varieties.

Chrome-nickel steels are much used for automobile shafts and gears, also for large axles and shafts. Rails and track bolts made of steel smelted from the Mayari ores of Cuba, which contain enough of the elements to form a low-chrome-nickel steel, have given very good service.

On account of the very fine case which these steels will take and the great toughness of the high-chrome-nickel they have been considerably used for heavy armor plate. With suitable heat treatment these steels are also used for medium armor plate, for protective decks and projectiles. For armor plate and projectiles a tenth of a per cent of vanadium is often introduced into the chrome-nickel steel.

**736. Chrome-vanadium steels**, one of the most recent developments among the alloy steels, have acquired extensive recognition on account of their excellent mechanical properties, uniformity in structure, sim-

\* *Steel and Its Heat Treatment*, p. 308.

plicity with which they can be heat treated, ease of machining and soundness of the cast metal. The composition, of chrome-vanadium steels used for structural purposes generally varies between the following limits: carbon, 0.20 to 1.0; chromium, 0.75 to 1.25; vanadium, 0.12 to 0.25; manganese 0.40 to 0.75 per cent, with silicon, phosphorus and sulphur low. For many structural purposes the carbon is kept between 0.20 and 0.30 per cent, the chromium is held in vicinity of 1.0 per cent with vanadium about 0.15 per cent.

Griffiths\* points out that chrome-vanadium steels have a higher critical range on heating and a lower range on cooling than ordinary steels of like carbon content. Also prolonged heating at high temperatures appears to have little deleterious effect. This sluggishness in structural transformation and capacity to withstand high temperatures renders these steels

comparatively easy to heat treat, a single quenching and drawing treatment being sufficient. For most purposes, chrome-vanadium steels, like the chrome-nickel, are heat treated.

The excellent strength and hardness and the high ductility of certain

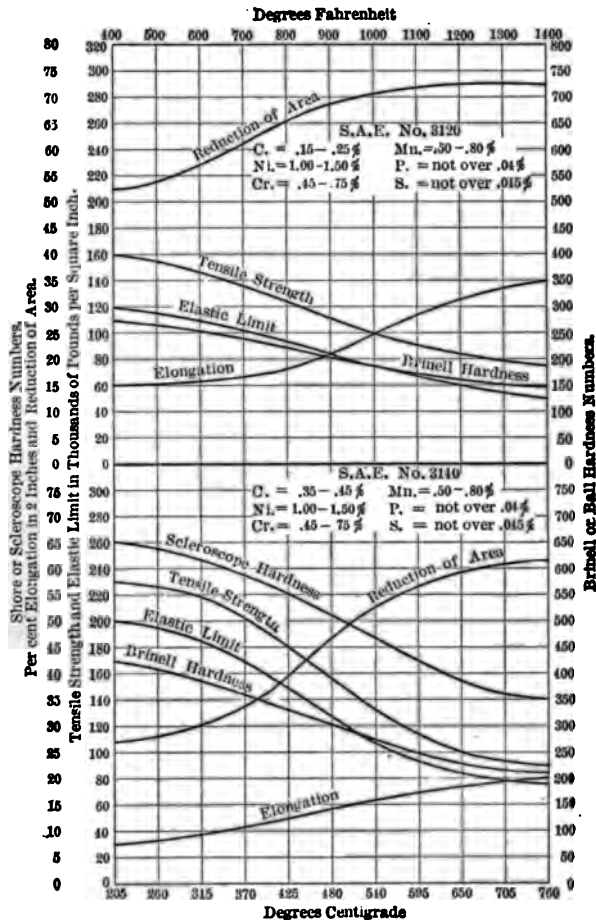


FIG. 5.—The Effect of Drawing Temperature on the Strength, Ductility and Hardness of Chrome-Nickel Steels.

Values are average minimum, excepting hardness values which are average, applying to round specimens  $\frac{1}{2}$  to 1 $\frac{1}{2}$  in. in diameter. No. 3120 were heated to 1585-1615° F., No. 3140 to 1485-1515° F., for 15 to 30 min., quenched in oil, reheated for 30 min. at temperatures indicated, and air cooled. (From Adopted Report of Standards Committee of Soc. of Automotive Engr. See *Trans.* Vol. 11, p. 18-20.)

\* *Proc. A. S. T. M.*, Vol. 17, p. 41.



of these alloy steels after different tempering treatments is typified by the results plotted in Fig. 6. The test-pieces used by Griffiths in these experiments were  $\frac{1}{2}$  in. in diameter over the standard 2-in. gage length with a total length of 18 in. With larger sections the strengths and hardness of steels of like compositions would be less and the ductilities greater.

Untreated chrome-vanadium steels are sometimes used for castings and shafting. For automobile springs and gears the steels are quenched and drawn. Chrome-vanadium steels may be readily case hardened, taking a glassy hard surface with a very fine-grained tough core. This property renders them valuable for dies and bearing raceways.

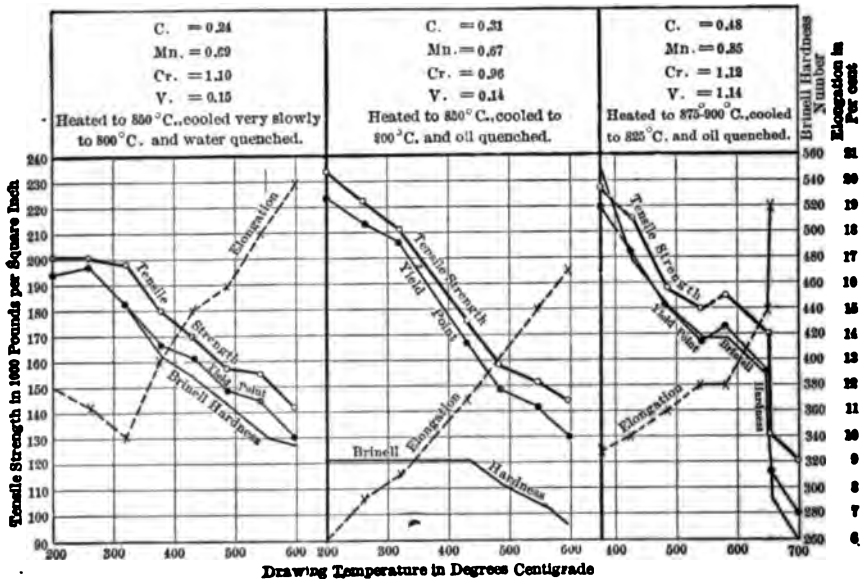


FIG. 6.—The Effect of Drawing Temperature on the Strength, Ductility and Hardness of Chrome-vanadium Steels. (Griffiths, *Proc. A. S. T. M.*, Vol. 17, p. 37.)

**737. High-speed Steels.**—One of the greatest advances in promoting rapid machining of metal parts has come about within the past twenty years through the introduction of steels for tools which can cut four or five times as rapidly as the simple carbon steels. Most of these steels can be run at a red heat without losing their hardness. In fact, many work to best advantage only when run at high temperatures and within a given range of cutting speeds.

Most high-speed steels contain from 15 to 20 per cent of tungsten, 3 to 5 per cent of chromium, 0.5 to 2.0 per cent of vanadium, 0.60 to 0.80 per cent of carbon with silicon, sulphur, and phosphorus running low. Some makers have used molybdenum to replace tungsten, but this prac-

tice seems to have been largely abandoned. More recently there has been a strong tendency to insert 3 to 5 per cent of cobalt in high-speed steels, the claim being made that cobalt increases the red-hardness of the steel. Although the critical points and the changes in constitution which the high-speed steels undergo in heat treatment is still largely a matter for further research to settle, there appears to be evidence,\* however, that at temperatures near the melting-point the carbon is in combination with the tungsten and chromium as a double carbide of these elements, but if the steel is slowly cooled the carbon combines with the iron at lower temperatures. What changes in constitution are effected by the introduction of vanadium is not known. It seems likely that it also forms carbides like chromium and tungsten.

By cooling these steels rapidly from a temperature near the melting-point, approximately 1250° C., down to a temperature below the ordinary critical range of the carbon steels, it is possible to retain the carbon in combination with the tungsten and chromium and to prevent the formation of the carbide of iron. Since these carbides impart great hardness and are very stable for all ranges of temperature up to a red heat, it follows that steels in which they are the essential components will retain their hardness at much higher heats than carbon steels in which hardness is conferred by the unstable carbide of iron.

The heat treatment generally given high-speed lathe and planer tools consists in heating to incipient fusion in a non-oxidizing atmosphere and quenching in an oil bath. For milling cutters and tools of accurate form, the quenching temperature is made slightly less to avoid injury to the shape of the tool. Where considerable toughness must be imparted, tools are quenched in molten lead and air cooled, or reheated in molten lead after oil quenching and air cooled. High-speed steels may be annealed by soaking for a long time at a temperature just above the critical range of carbon steels. After such treatment they possess high strength and good ductility.

Aside from tools, high-speed steels are also used in parts which must withstand high heat and wear, as in the exhaust valves of gas engines, and in dies through which brass is extruded.

\* Carpenter, H. C. H. in *Jour. Iron and Steel Inst.*, 1905, Pt 1, p. 433; also Edwards, C. A., in *Jour. Iron and Steel Inst.*, 1908, Pt. 2, p. 104.

## CHAPTER XXV

### CAST IRON AND MALLEABLE CAST IRON

#### CAST IRON

**738. Importance of Cast Iron.**—On account of cheapness, strength, ease with which it may be melted and cast into more or less intricate shapes, ease of machining, and ease with which its hardness may be varied, cast iron is the most-used of the cast metals employed in engineering constructions and machines. It is very extensively fabricated into water-pipes, cylinders, car-wheels, agricultural machinery, stoves, hardware, machine frames, bed plates, and column bases; and to a lesser extent for columns, grate bars, ornamental castings, pipe fittings, and agricultural implements. Further uses are indicated in Table 1. Where toughness is necessary cast iron is displaced by the more expensive malleable cast iron or by the still more costly cast steel. Again in constructions where the metal must withstand corrosion, brasses, bronzes and other alloys, all of which are very much more expensive than cast iron, displace it.

As an indication of the great use of cast iron, we note that about one-fifth of the pig-iron annually made in the United States is remelted and made into cast iron. From this pig iron and about a million tons of scrap approximately seven million tons of gray-iron castings are produced.

#### MANUFACTURE OF CAST IRON

**739. Remelting of Pig Iron.**—Although pig iron from the blast furnace is sometimes molded into final form, most of the pig iron used for castings is remelted before being molded into final shape. Remelting is necessitated by the variability in the pig iron run from a given furnace, by the difficulty of adjusting the composition of the molten iron, and by the necessity of mixing different grades of pig iron in order to secure the desired grades of castings.

Most of the ordinary gray iron used in machine parts is remelted in the cupola, the better grades of gray iron—often called gun iron—and the white iron used in making malleable cast iron are generally remelted in the air-furnace. Some use has also been made of small open-hearth furnaces to remelt pig iron for high-grade cast iron and malleable cast iron.

The function of the remelting furnace is simply to produce homogeneity in the charge which has been proportioned with reference to the use of the product. Changes between the average composition of the metal charged and that of the castings are, in general, small although sometimes important.

**740. Materials Charged.**—Compositions of the pig irons commonly used in smelting cast iron are given in Art. 582. In many parts of the country the chemical analysis of the pig iron serves as means of grading it and purchase is made on this basis. Generally the silicon content is specified and the sulphur limit prescribed. Some foundrymen still rely on the character of fracture exhibited by the pig iron as a criterion of composition. Special pig irons containing high percentages of silicon, the Scotch irons and ferro-silicons, are sometimes added to soften the iron; while others containing high percentages of manganese, ferromanganese, for example, are used as hardeners.

Besides pig iron, from a tenth to a half of the metal charged consists of the refuse from previous heats and whatever old machinery or parts of cast iron the foundrymen can purchase. All of this heterogeneous mass is termed scrap. On account of variability in its composition the proportion of scrap is generally less than 25 per cent of the metal charged when the best grades of castings are being made.

Coke is the fuel most commonly used in the cupola, bituminous coal in the air-furnace. Under favorable conditions with large furnaces the ratio of fuel to iron is about 1 : 8 or 1 : 10 for the cupola and 1 : 4 for the air-furnace.

A flux consisting of crushed limestone or other form of lime carbonate is sometimes added in very small amounts to slag off the earthy impurities and reduce the sulphur content of the cast iron.

**741. The cupola** is a sort of small blast furnace. It consists of a vertical cylindrical steel shell of nearly uniform diameter lined with fire-brick. Fig. 1 shows an elevation of a cupola. At the bottom is placed the hearth or crucible which extends upward a short distance to the level of the tuyeres. At the bottom of the hearth is located the taphole and opposite

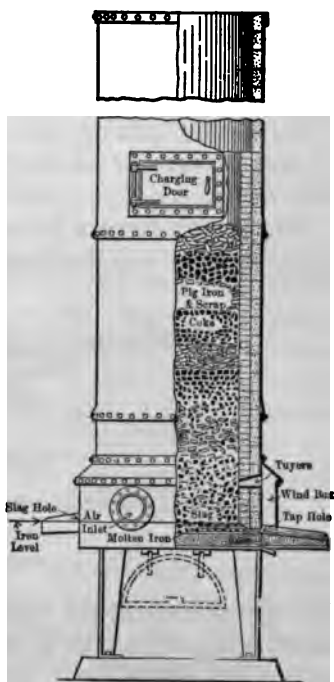


FIG. 1.—Cupola.

to it several inches above the bottom is the slag-hole. The tuyeres are placed in one or two circumferential rows. Air at a pressure of 1 lb. per square inch or less is served to the tuyeres through the wind box which surrounds the hearth. Above the tuyeres is the melting zone which is surmounted by the stack, a door for charging being placed in the latter. Cupolas vary in internal diameter up to 10 ft. The common sizes are from 4 to 6 ft. in diameter with their charging doors 12 to 25 ft. above the hearth bottom. Cupolas of these sizes will run from 10 to 20 tons of metal per hour.

In operating a cupola, kindling is first placed on the hearth and a thick layer of coke on top of it. Alternate layers of pig iron mixed with scrap and layers of fuel are then dumped in until the stack is filled to the level of the charging door. If flux is used it is charged immediately after each layer of metal. After the fire has been kindled and the bed of fuel well ignited the blast is turned on, and in about ten minutes molten iron trickles from the tap-hole. The tap-hole is then closed with a plug of fire-clay. As the slag accumulates it runs off through the slag-hole and from time to time the iron is tapped into a large ladle. Ordinarily a cupola is charged and discharged several times a day, but at some plants they are run continuously for several days. The necessity of repairing the lining at frequent intervals prohibits long runs.

During the melting process a small amount of iron and 0.2 to 0.3 per cent of silicon is oxidized, while 0.05 to 0.10 per cent of sulphur and, under certain conditions, a little carbon is absorbed from the fuel.

**742. The air-furnace** is somewhat like the puddling furnace used in making wrought iron, but larger in size (Fig. 2). At one end it is provided

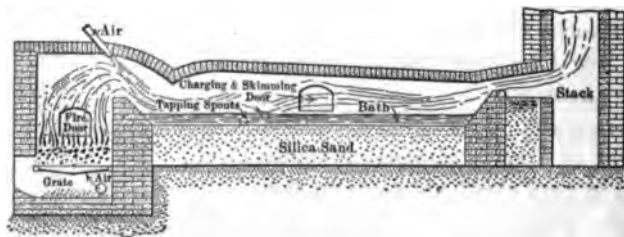


FIG. 2.—Showing Principle of Air-furnace.

with a fire-box and at the other a stack; between is a shallow rectangular hearth, served by a door in the side. Separating the hearth and the fire-box is a low vertical wall called the fire-bridge. The opposite end of the hearth is formed by the flue-bridge which rises to the same height as the fire-bridge. Air-furnaces range in capacity from 5 to 40 tons.

Before the charge is introduced into the furnace the bottom of the hearth is covered with an even layer of sand; then the scrap and pig iron are

placed upon it. The furnace door is sealed with clay and the fire started. Melting of the iron is accomplished principally through radiation from the sloping roof and, in part, through the heat of the hot gases which sweep over the hearth. During the melting process the bath is rabbled occasionally with iron bars to promote uniformity in melting and in composition. If much slag forms on the bath it is partially skimmed off to raise the temperature and promote oxidation of the carbon and silicon. In order that the top metal, which is the hottest, may be run from the furnace, tapping is done through a set of holes placed at different elevations in the hearth. The usual rate of producing iron with the air-furnace is 3 to 4 tons per hour.

**743. Comparison of Cupola and Air-furnace Processes.**—The cupola process is quicker, cheaper in installation and in operation; the metal is hotter and more uniform in temperature—conditions which mean much in casting; the loss of metal through oxidation is also less than in the air-furnace. On the other hand the air-furnace produces a larger quantity of high-grade iron at a single tapping. The metal in the air-furnace not being in contact with the fuel absorbs neither sulphur nor carbon. The air-furnace process is under better control and permits better regulation in the composition of the iron. In neither process, however, has there been a successful attempt at utilization of the latent heat in the escaping gases.

## MOLDING

**744. Patterns of the castings** are made either of wood coated with shellac or metal. Wood is largely used when only a few castings are desired. When the number of castings is to be great, it is preferable to use a brass pattern and thus avoid imperfections likely to be found in the castings due to damaging of wooden patterns. If a number of castings of similar shape are to be made it is customary to join several patterns in such way that they may be poured simultaneously. Such patterns are said to be gated. Patterns are always made larger than the casting to allow for contraction in cooling. For ordinary gray cast iron an allowance of an eighth-inch per foot is a common rule. White cast iron and steel shrink double this amount; brass and copper about 50 per cent more and lead and zinc two and one-half times as much as gray cast iron. The fundamental considerations in designing a pattern are: (1) make the pattern of such shape that it may be removed from the sand without damaging the mold, if of intricate shape it may be necessary to make the pattern in two or more parts; (2) use fillets of large radius at all sharp angles and corners and thus avoid planes of weakness arising from the crystalliza-

tion of the metal; \* (3) avoid joining heavy and light sections wherever possible, since these parts, cooling at unequal rates, will be highly stressed at their junction (if such design is necessary some provision should be made for rapidly cooling the heavy section); (4) when possible avoid shapes where the ends of the casting will be rigidly held by the mold and contraction stresses or checking thus produced in the intermediate parts of the casting.

**745. Cores.**—When it is necessary to make a hollow casting some sort of core is used. This is located in proper place in the mold by projecting fins which rest in core prints that are made by corresponding projections of the pattern. If the core is of such size or shape that it cannot be held in place by core prints, iron supports called chaplets are placed in the mold. Generally cores must be so made that they will offer little resistance to shrinkage of the metal and will not burn on to the iron. They are commonly molded of dry sand mixed with flour, molasses, linseed oil or patented compound and baked in an oven. Sometimes green sand cores are used. When it is particularly necessary to avoid all stressing of the casting, cores are made with centers of crushed coke or of pipe wound with hay rope, the outside being of sand. Cores are vented to permit the escape of gases generated in pouring.

**746. Materials for Molds.**—Not only must the material composing the mold retain the metal and give it a smooth and true surface but it must also be sufficiently porous to allow the escape of air and gas. Sand is the most refractory cheap material for this purpose. A good molding sand generally contains not less than 80 per cent silica and 5 to 10 per cent of alumina. The higher the temperature of the molten metal, the more refractory must the sand be. Consequently the silica content must be higher for steel than for cast-iron castings. Some alumina, however, is needed to furnish requisite cohesiveness in the sand. Small proportions of magnesia, lime and iron oxide are usually present also in molding sands. More than 2 or 3 per cent of the carbonates is objectionable owing to the gas formed in calcination of them, while metallic oxides render the sand less refractory. Another important factor which affects the cohesiveness and the porosity of molding sand is the gradation in sizes of particles. The smoothness of the surface of the casting is also affected by the fineness of the sand grains. Therefore, molding sand is generally screened through a 20-mesh sieve, the gradation in sizes below that opening being determined by the work in hand.

\* When a metal cools the crystals form perpendicularly to the surfaces. If a corner is left sharp there will be a plane of separation between crystals bisecting the angle at the corner. Since the cohesion across these planes is less than in the crystals, a pronounced weakness is thus produced. Fig. 3 shows such planes for a square bar. By rounding the corners interlocking of the crystals is promoted and a stronger, more homogeneous casting is obtained.

Parting sand is a highly refractory sand which cannot be rendered cohesive by the addition of water.

Facing materials are shaken over the pattern or surface of the mold to prevent the sand from being burnt and to make the castings leave the mold freely, thus preserving a bright surface and avoiding expense in cleaning. Fine soft coal, graphite, charcoal and talc are among the substances which are mixed with six to fifteen times as much fine molding sand to make facings.

Loam is the name for a soil carrying a high content of silicious sand, considerable clay and more or less decayed vegetable and animal matter. The term is also used to designate artificial mixtures made of sand and clay with some sawdust or rye meal.

**747. Molds.**—The kind of castings and the number, size and shape determine the character of the molds. The common types of molds are green-sand, dry-sand and loam molds. Besides these, cast-iron molds are considerably used when a permanent type of mold is desired.

*Green-sand Molds.* The pattern is surrounded by a flask of wood or cast iron which serves to hold the sand in place. Generally the flask consists of two, sometimes three or more, rectangular frames of equal size which may be doweled or locked together to form a bottomless box. In molding many simple objects, half of the pattern is bedded in the lower frame, called the drag, and the other half is covered by the upper part of the flask, called the cope. The procedure is to place the drag on a mold board bottom-side up with the pattern or portion of the pattern resting on the mold board. Molding sand moistened with sufficient water to render it coherent is rammed about the pattern and several small vent holes are punched through to the pattern. A bottom board is put over the drag and the drag is turned over. The remainder of the pattern is inserted and a bit of parting sand is sifted over the top of the drag, the pattern being kept clean. Next, the cope is put in place and plugs for the runner \* and riser \* are properly located. Molding sand is tamped about the pattern and vents provided. The plugs are then withdrawn, the cope carefully lifted and gates from plugs to pattern are cut. A draw spike is attached to the pattern, and tapped until the pattern can be removed. After the loose sand has been removed a facing may be smeared on the surfaces of the mold; the cope is then placed on top of the drag and locked in position for pouring.

Green-sand molds are much used, especially for articles of like shape, because they can be rapidly made at low cost. When there is

\* A runner is the canal through which the molten metal is poured. It is provided with a basin at the top and a hole at the bottom connecting with the mold called a gate. A riser is a vertical canal leading from the mold to the top of the cope. It serves to vent the mold, supply metal as the casting cools, and to carry off dirt.



a large demand for a certain type of casting, molding machines are frequently used.

*Dry-sand molds* are fashioned in iron flasks in much the same way as those of green sand. A rather coarse loamy sand is used and the molds are dried at 300 to 400° F. After baking, the surface of the mold is generally coated with a wet mixture of graphite, or charcoal with clay. Such molds are strong, will withstand hard usage, and, if properly vented, produce sound smooth castings. Dry-sand molds are well adapted for the production of cylinders, rolls, engine beds and other heavy castings where the pressure of the metal is great, or where a smooth wall of uniform thickness must be obtained.

Dry-sand molds are likely to be somewhat distorted during drying and cause heavier shrinkage stresses in castings than do green-sand molds. On the other hand, castings made in dry-sand molds are more sound, smoother, and freer from inclusions of sand and dirt than those cast in green sand.

*Loam molds* are used principally for very large castings which are bounded by surfaces of revolution. Castings of heavy engine cylinders and fly-wheels are generally made in loam molds. The outer casing of the mold is ordinarily built of brick, sometimes of iron. To the interior of this casing dampened mixtures of loam are plastered. The surface of the mold is generated by revolving a sweep, the end of which is fashioned in conformity with the surface desired for the casting. The mold is then baked and faced. Owing to the rigidity of loam molds, provision is made during the fabrication to permit rapid destruction of certain parts immediately after the casting is poured in order that the latter may contract freely.

**748. Chills.**—Surfaces of castings which are subjected to heavy wear are made hard by rapidly cooling them in chills. Chills consist of pieces of cast iron which form the surface of the mold in contact with the part of the casting to be hardened. Sticking of the iron to the chills is prevented by coating the latter with shellac and plumbago or with a thin film of light oil. To avoid explosions resulting from contact between the molten metal and the chill, it is necessary to heat the latter to 300 or 400° F. before pouring. The treads of car wheels and the bearing surfaces of rolls are cast against chills. Changes in composition due to chilling are explained in Art. 751. Fig. 3 shows the effect of chilling on structure. The depth in inches of white iron produced by chilling is termed chill.

**749. Cleaning Castings.**—Flasks are removed as soon as the castings have solidified, but the pieces are generally allowed to remain in the sand until cool. Sand and dirt adhering to the surfaces of the castings are removed by rattling, by pickling in acid or by sand blasting. Rattling is done in a device similar to that used in testing paving brick. (Appendix

A.) It can only be successfully employed on sturdy regular-shaped pieces. Rattling also produces a hard skin on the castings. Pickling is used on fragile or intricate castings and on pieces which must be



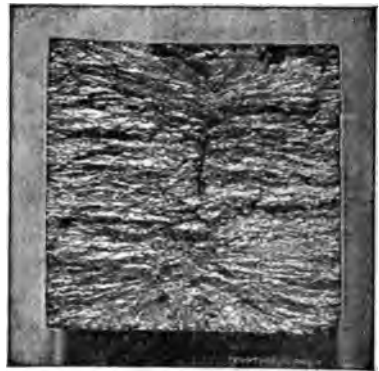
(a) Sand-mold, not Chilled.



(b) Chilled on One Side.



(c) Chilled on Four Sides.



(d) Chilled on Opposite Sides.

FIG. 3.—Showing the Effect of Chilling on Car-wheel Iron. (*Jour. Frank. Inst.*, April, 1897.)

machined. Dilute solutions of hydrochloric or sulphuric acid are often used for pickling, but they attack the iron. Hydrofluoric acid in a 5 per cent solution is more efficacious since it attacks the sand. Sand blasting is an effective means of cleaning heavy work.

## COMPOSITION AND CONSTITUTION

**750. The Principal Constituents.**—Cast iron is really unrefined steel contaminated with larger proportions of impurities. As in the carbon steels, the five main impurities in cast iron are carbon, silicon, sulphur, phosphorus and manganese. They constitute from 5 to 8 per cent of cast iron, by weight. Besides these elements oxygen, nitrogen and copper are often present in small percentages, while, to irons used for certain purposes, vanadium and titanium are sometimes added.

Since the properties of cast iron are dependent principally upon the proportion of the different impurities and the combinations which they form in the iron, the influence of each of the essential constituents will be considered. Owing to the variety of forms and combinations in which carbon is found in cast iron (see Art. 659 and 662), the influence of this element is of the most importance in determining the properties and the value of the iron. Thus we find three main types of cast iron—gray, white, and mottled iron—which owe their characteristics and properties to variations in the form of carbon content.

**751. Carbon in Cast Iron.**—The proportion of carbon in cast iron generally lies between  $2\frac{1}{2}$  and 4 per cent by weight. Carbon, as has been shown in Art. 662, occurs in two forms—either free as graphite, or chemically combined with the iron as iron carbide ( $\text{Fe}_3\text{C}$ ). When free, the proportion of the volume of the metal occupied by the graphite, due to its specific gravity being much less than that of iron, is about 3.5 times as much as the percentage by weight. Therefore the spaces occupied by the flakes of graphite aggregate 8 to 14 per cent of the bulk of the iron. When the carbon is combined with the iron the proportionate bulk of the iron carbide is still greater, being over fifteen times the percentage of combined carbon.

The amount of carbon which the molten iron will retain in solidifying is dependent upon the composition of the iron. Again the proportion of the carbon which is retained in combined form is influenced greatly by the presence of other elements and by the rate of cooling. Thus by adding manganese or chromium the solubility of carbon in iron is increased and much of it is combined with the iron. On the other hand, additions of silicon or aluminum reduce the solubility of carbon in iron and promote the formation of graphite. Rapid cooling tends to cause combined carbon, slow cooling furthers increase in graphite. When the composition is properly adjusted it is possible to retain the carbon as carbide of iron by rapid cooling or to precipitate it by slow cooling.

If in cooling the carbon is largely precipitated, more or less uniformly in the form of graphite flakes, the iron is soft and presents a dull gray fracture which is brightened here and there by glistening iron crystals.

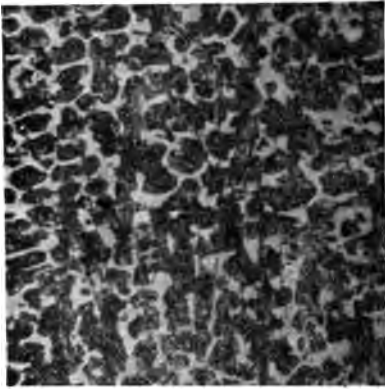
This is **gray cast iron**. When the carbon is retained in combined form, the cast iron is very hard and brittle; and the silvery white fracture has led to the name, **white cast iron**. In some irons the major portion of the carbon is retained in the combined form while a lesser part is precipitated as graphite. Such irons exhibit a white fracture spotted with dark



(a) Gray Cast Iron with finely divided graphite more or less globular in form ( $\times 50$ ). (Aston.)



(b) Gray Cast Iron with graphite in large flakes ( $\times 50$ ). (Harbison Walker Refractories Co.)



(c) White Cast Iron. The white constituent is cementite and the dark one is pearlite ( $\times 100$ ). (Storey.)



(d) White Cast Iron under higher magnification ( $\times 500$ ); showing lamellar pearlite (banded structure), flakes of graphite, and cementite. (Wüst.)

FIG. 4.—Photomicrographs of Gray and White Cast Irons.

gray patches and are termed **mottled cast irons**. Photo-micrographs of gray and white cast irons appear in Fig. 4.

From the foregoing it follows that irons containing large amounts of manganese or chromium are likely to be permanently white while those having a high silicon content are gray. With proper adjustment in composition, cast iron may be rendered white by cooling rapidly or gray by cooling slowly from the molten state.

The proportion of carbon and its form influence more or less most of the physical and mechanical properties of cast iron. The fusibility varies inversely with the carbon content and inversely with the percentage of combined carbon.\* Therefore white cast iron has a lower melting-point than gray iron, but it is less fluid when molten. Shrinkage varies inversely as the carbon content, but white iron shrinks nearly twice as much as gray iron. The specific gravity of gray iron increases with the decrease in carbon content and varies from 425 to 450 lb. per cubic foot. White cast iron is heavier than gray due to the chemical combination of the carbon and weighs in the vicinity of 475 lb. per cubic foot.

The influence of the proportion of combined carbon on the constitution and mechanical properties of cast iron containing 4 per cent of carbon is very well illustrated by Howe's diagram in Fig. 5.† It will be noted that the tenacity of the iron (the whole) is governed by the strength of the matrix which is a maximum when the matrix contains 1.2 per cent combined carbon. The hardness also varies with the hardness of the matrix and increases directly with percentage of combined carbon (cementite). Fig. 5 also brings out the effect of variations in the proportion of combined carbon on the machining qualities and general uses of cast iron. Similar diagrams might be worked out for irons with less total carbon. In such cast irons, the weakening effect of the graphite will be less and the influence of the properties of the matrix will be still more pronounced; therefore, the points *G*, *A* and *F* will be higher.

The size and shape of the graphite plates also exercise an important effect upon the strength of the cast iron. Large thin plates of graphite weaken the iron much more than small round grains (see Fig. 4).

**752. Silicon in Cast Iron.**—Next to carbon, silicon exercises the most important influence on the properties of cast iron. It combines with the iron, forming iron silicide, which in turn is dissolved in the ferrite. The proportion of silicon in cast iron usually runs between 0.5 and 3.5 per cent, although certain special castings for acid containers are made with a much higher silicon content. Silicon in small percentages increases the fluidity of the molten iron, decreases blow-holes and increases the density of castings. Silicon also reduces the solubility of carbon in iron. According to Wust and Peterson each per cent of silicon throws out of solution 0.27 per cent carbon. Furthermore, since silicon indirectly promotes the decomposition of hard cementite into soft ferrite and graphite ‡

\* From results given by Porter in *Trans. Am. Foundrymen's Assoc.*, Vol. 19, p. 113, the following equation is derived:  $F = 2175 - 62.5C$ , where  $F$  = melting temperature in  $F.^{\circ}$  and  $C$  = per cent combined carbon (less than 3.5).

† *Proc. A.S.T.M.*, Vol. 2, p. 252.

‡ Porter claims that the insertion of 1 per cent of silicon into cast iron effects a pre-precipitation of 0.45 per cent carbon as graphite. (Previous citation.)

it acts as a softener and also decreases shrinkage. However, when present in excess of the amount required to decompose cementite, the direct hardening influence of iron silicide becomes noticeable; and with 5 or 6 per cent of silicon the iron is hard and has a mirror-like fracture. In short, by varying the silicon content the foundryman exercises a most important control over a wide range in the properties of cast iron.

The combined influence of carbon and silicon upon the structure of cast iron is well illustrated in Fig. 6.\* The manganese in all of the test

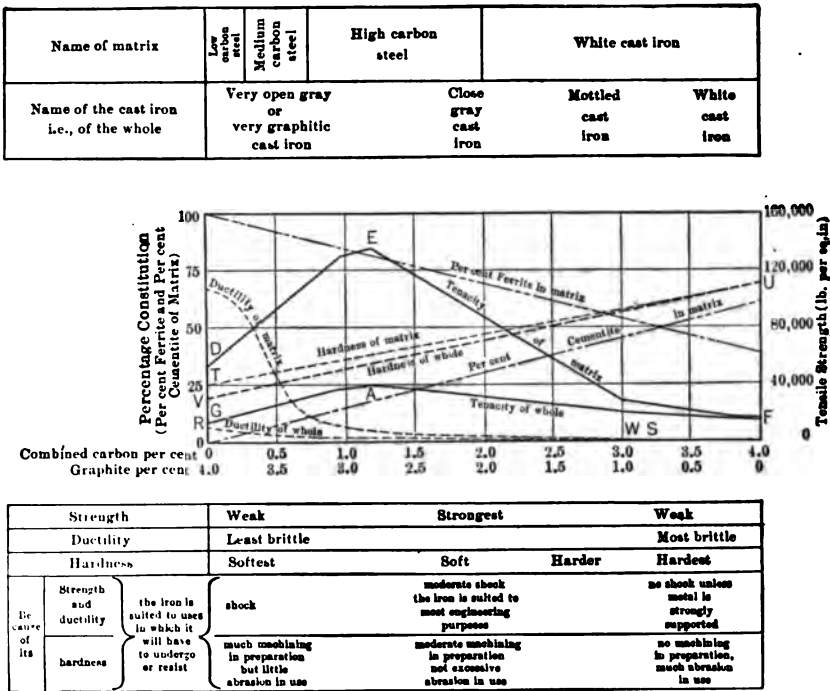


Fig. 5.—Influence of Carbon on Constitution and Properties of Cast Iron. (Howe.)

heats ranged from 0.25 to 0.35; sulphur averaged about 0.06 and phosphorus from 0.13 to 0.18 per cent. Cylindrical specimens 2 in. in diameter and 12 in. long were poured in green-sand molds with proper regard for disturbing influences due to variations in pouring temperatures. They were cooled in the molds two hours before quenching in water. In the diagram, the line *G-G* represents limiting combinations of silicon and carbon at which the test-pieces were gray. The line *M-M* shows the varying combinations of carbon and silicon producing a mottled fracture, while the line *W-W* represents the boundary for white fractures. For smaller

\* G. M. Thrasher in *Met. and Chem. Engr.*, Vol. 13, p. 40.

test-pieces or for more rapid rates of cooling these lines would be shifted upward to the left; for larger test-pieces or for slower cooling the lines would be displaced toward the lower right corner of the diagram.

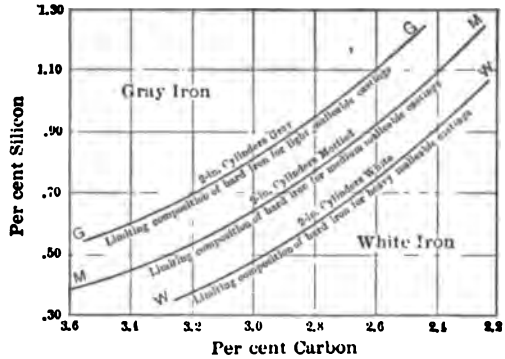


FIG. 6.--Relation of Carbon and Silicon Contents to Structure of Cast Iron under Normal Cooling. (Thrasher.)

Turner, Associate of Royal School of Mines, England. The hardness tests were made by a sclerometer, the weight necessary to produce a scratch of given width with a standard diamond point being the criterion of relative hardness. Fig. 7 shows that maximum hardness and crushing strength obtained when the castings were rendered sound and the per cent graphite was a minimum, per cent silicon being about 0.80 per cent for the iron tested. Further increase in silicon to 1 per cent produced maximum stiffness, when increased to 1.4 per cent maximum transverse strength resulted, at 1.8 per cent the greatest tensile strength obtained, and at 2.5 per cent silicon the maximum softness. Increasing the silicon beyond 2.5 per cent caused an increase in hardness and brittleness.

The influence of silicon on the mechanical properties of irons containing about 2 per cent total carbon is well shown by Fig. 7. These experiments were very carefully made on specially prepared irons under the direction of Thos.

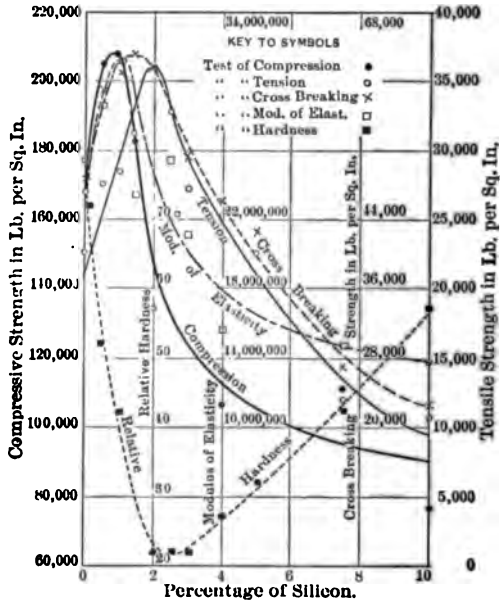


FIG. 7. -Influence of Silicon on Mechanical Properties of Cast Iron. Specimens were  $1\frac{1}{2}$  in. in diameter and were tested with skin on. (From Turner's *Iron*.)

The very important influence of silicon in reducing shrinkage is well

brought out in Fig. 8, showing results of W. J. Keep. From other data by Keep, Fig. 9, which shows the variation in transverse strength for different sizes of bars and different silicon contents, has been prepared. The data emphasize the necessity of testing bars of the same thickness as the finished casting, if a knowledge of the strength of the metal in the casting is wanted.

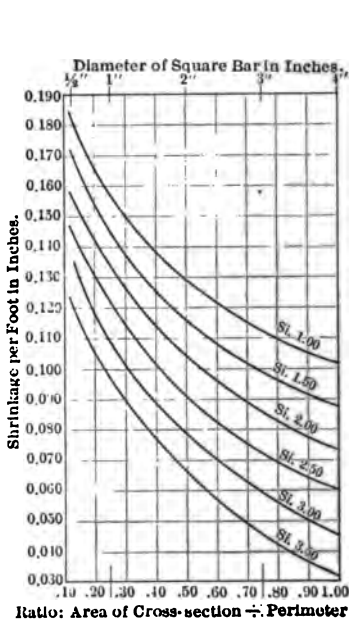


FIG. 8.

FIG. 8.—Influence of Silicon on Shrinkage of Cast Iron Specimens of Various Areas of Cross-section. (Keep.)

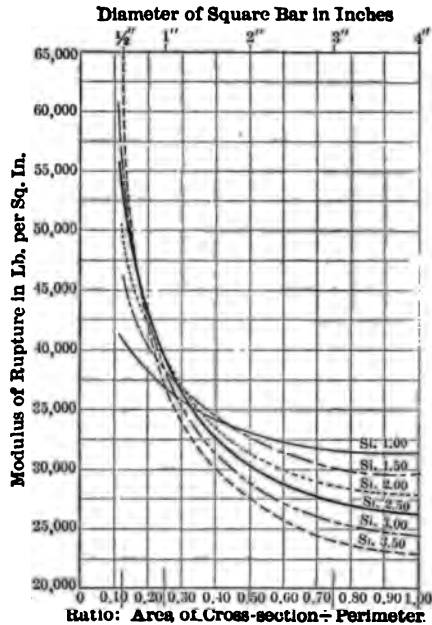


FIG. 9.

FIG. 9.—The Variation in Cross-breaking Modulus of Rupture of Cast Iron for Different Sizes of Bars and for Varying Percentages of Silicon. (Keep.)

**753. Sulphur in Cast Iron.**—Sulphur is an undesirable element in cast iron and is generally limited to less than 0.1 per cent. Since it is believed to promote the formation of combined carbon the above limit is often doubled on irons cast in chills. It combines with manganese to form the sulphide ( $MnS$ ) or, if the manganese is very low and not sufficient to satisfy the sulphur, iron sulphide ( $FeS$ ) may be formed. Since these sulphides solidify at considerably lower temperatures than cast iron they tend to make castings brittle and weak at high temperatures. Sulphur in high percentages (0.5 per cent or over) also increases shrinkage and causes hard, brittle iron. These evil effects may be neutralized by proper additions of silicon.



**754. Phosphorus in Cast Iron.**—According to Stead, phosphorus occurs in gray iron in a eutectic of iron phosphide plus iron, and in white iron in a eutectic of iron carbide, iron phosphide and iron. Since phosphorus in these eutectics is under 10 per cent, a small proportion of the element generates a large amount of this alloy. When phosphorus does not exceed 0.5 per cent it has no marked effect upon cast iron. If more than 2 per cent of phosphorus is present the iron is embrittled and the strength diminished. High-phosphorus irons are somewhat more fluid and shrink much less than irons low in phosphorus. High-phosphorus irons, therefore, take a good impression of the mold and are much used in making thin stove castings and ornamental castings where great strength and toughness are not essential.

**755. Manganese in Cast Iron.**—The proportion of manganese ordinarily found in cast iron ranges from 0.1 or 0.2 up to 2 per cent. When present in such proportions manganese combines with sulphur, forming manganese sulphide ( $MnS$ ), and—having satisfied sulphur—with carbon to form manganese carbide ( $Mn_3C$ ). The latter is found in cementite united with iron carbide. Ferromanganese is often added to the molten iron to reduce the sulphur and oxygen contents. This is accomplished by combination and partial withdrawal of the oxides and sulphides of manganese into the slag. Manganese increases the solubility of carbon in iron and opposes the liberation of graphite. Increased shrinkage and hardness are promoted by increasing the manganese content beyond that required to satisfy sulphur. Therefore manganese must be kept low in gray iron which is to be machined, while in parts which must withstand abrasion a high manganese content is desirable. The effect of manganese on strength is not material.

**756. Other Elements in Cast Iron.**—Copper is found in some ores, and when under 1 per cent appears to make cast iron more dense. About a tenth of 1 per cent of titanium or vanadium is occasionally added to the molten iron while in the ladle to cleanse it of oxygen. Such additions have in some instances produced considerable increases in strength. The high cost of these metalloids has, however, prohibited extensive use. Aluminum has much the same effect on cast iron as silicon, but being more expensive has been little used. Nickel and chromium have been added to molten cast iron but no beneficial results appear to accrue from additions of less than 1 per cent of either element.

**757. Defects in Cast Iron.**—Checks, segregation, blow-holes and coarse grain, the principal defects in cast iron, originate during the cooling of the castings. Checks are small parallel cracks in the surface of a casting. They generally run transverse to the long axis of the piece. Checks may arise from errors in designing the shape of the casting or mold which prevent contraction during cooling. Irons of high sulphur content are

liable to have this defect owing to their great shrinkage and lack of strength while at a red heat.

Segregation is very pronounced in high phosphorus irons, where the eutectics of iron phosphide and iron separate from the main part of the metal and form brittle masses which are more or less well connected, depending on the amount of phosphorus present. Even with smaller percentages of phosphorus there appears to be a well-marked tendency to the formation, here and there, of little knots of metal which are found attached within gas cavities. Analysis of the knots show that the phosphorus and sulphur contents are very much above the mean compositions. The sulphides are also found in greatest proportion in the top of the casting and in the parts cooled most slowly. Carbon and silicon sometimes segregate in such manner that interior portions of the metal are white and exterior parts are gray. When such segregations occur at different parts of the surface of a casting, they render it very difficult to machine. Sometimes relief of the non-uniformity can be had by annealing the piece.

Segregations make it very difficult to secure representative samples of castings for chemical analysis. Since sulphur segregates on the top of a casting, drillings should be made through the castings from the top and the sample thoroughly mixed. Variations in size of section will cause changes in the combined carbon content; more combined carbon will exist in the skin than in the interior; these conditions must all be kept in mind in locating drill holes for borings or in cutting samples from the casting.

Blow-holes are generally due to improper venting of the mold or to a high proportion of sulphur. If pronounced, they seriously affect both strength and toughness of the casting.

A coarse or open grain in the iron is caused by too slow cooling, or it may be due to a very high phosphorus content. In thick parts a coarse open grain is generally found near the center of the section (see Fig. 2a) and is quite difficult to prevent. A more compact structure is often gotten by lowering the silicon content or by charging turnings or chips of cast iron along with the pig iron.

Besides the above-mentioned defects, spongy spots and "cold shuts" sometimes result from lack of fluidity in the iron or from improper gating. Cold shuts are fault planes in the metal produced by the solidification of part of the casting before the remaining molten metal was run into place. Spongy spots are exaggerated forms of open grain; they are often due to a solidification of metal in the risers before the interior of the casting has solidified. The interior is thus cut off from the supply of metal which is needed to fill voids caused by shrinkage in cooling, and a porous structure results.

**758. Compositions Suitable to Different Kinds of Castings.**—From a study of the compositions of castings used for different classes of work Table 1 has been prepared.\*

**TABLE 1.—SUGGESTED COMPOSITIONS FOR CAST IRONS SUBJECTED TO VARIOUS USES. (PORTER)**

Kind of Castings.	PER CENT COMPOSITION.					
	Silicon.	Sulphur.	Phosphorus.	Manganese.	Combined Carbon.	Total Carbon.
Acid resisting.....	1.00-2.00	< .05	< .40	1.00-1.50	.....	3.00-3.50
Agric. machinery, ordinary.....	2.00 2.50	.06-.08	.60-.80	.60-.80		
Agric. machinery, very thin.....	2.25-2.75	.06-.08	.70-.90	.50-.70		
Air cylinders.....	1.00-1.75	< .09	.30-.50	.70-.90	.....	3.00-3.30
Ammonia cylinders.....	1.00-1.75	< .09	.30-.50	.70-.90	.....	3.00-3.30
Automobile.....	1.75-2.25	< .08	.40-.50	.60-.80		
Automobile cylinders.....	1.75 2.00	< .08	.40-.50	.60-.80	.55-.65	3.00-3.25
Brake shoes.....	1.40 1.60	.08-.10	.30	.50-.70	.....	low
Car, gray iron.....	1.50 2.25	< .08	.40-.60	1.60-.80		
Car wheels, chilled.....	.60-.70	.08-.10	.30-.40	.50-.60	.60-.80	3.50-3.70
Chilled.....	.75-1.25	.08-.10	.20-.40	.80-1.20		
Collars and couplings for shafting.....	1.75-2.00	< .08	.40-.50	.60-.80		
Crusher jaws.....	.80-1.00	.08-.10	.20-.40	.80-1.20		
Dynamo and motor frames, base and spiders, large.....	2.00-2.50	< .08	.50-.80	.30-.40	.20-.30	low
Dynamo and motor frames, base and spiders, small.....	2.50-3.00	< .08	.50-.80	.30-.40	.20-.30	low
Electrical.....	2.00 3.00	< .08	.50-.80	.30-.40	.20-.30	low
Fly wheels.....	1.50 2.25	< .08	.40-.60	.50-.70		
Gas engine cylinders.....	1.00 1.75	< .08	.20-.40	.70-.90	.....	3.00-3.30
Gears, heavy.....	1.00 1.50	.08-.10	.30-.50	.80-1.0	.....	low
Gears, small.....	2.00-2.50	< .08	.50-.70	.60-.80		
Grate bars.....	2.00 2.50	< .06	< .20	.60-1.00	< .30	low
Gun iron.....	1.00-1.25	< .06	.20-.30	.....	.80-1.00	low
Heat resistant iron.....	1.25 2.50	< .06	< .20	.60-1.00	< .30	low
Hydraulic cylinders, heavy.....	.80-1.20	< .10	.20-.40	.80-1.00	.....	low
Hydraulic cylinders, medium.....	1.20-1.60	< .09	.30-.50	.70-.90	.....	low
Locomotive, heavy.....	1.25 1.50	< .08	.30-.50	.70-.90		
Locomotive, light.....	1.50 2.00	< .08	.40-.60	.60-.80		
Locomotive cylinders.....	1.00 1.50	.08-.10	.30-.50	.80-1.00		
Machinery, heavy.....	1.00-1.50	< .10	.30-.50	.80-1.00	.....	low
Machinery, medium.....	1.50-2.00	< .09	.40-.60	.60-.80		
Machinery, light.....	2.00 2.50	< .08	.50-.70	.50-.70		
Pipe fittings.....	1.75-2.50	< .08	.50-.80	.60-.80		
Pipe fittings for superheated steam.....	1.50-1.75	< .08	.20-.40	.70-.90	.....	low
Pulleys, heavy.....	1.75-2.25	< .09	.50-.70	.60-.80		
Pulleys, light.....	2.25-2.75	< .08	.60-.80	.50-.70		
Railroad.....	1.50-2.25	< .08	.40-.60	.60-.80		
Rolls, chilled.....	.60-.80	.06-.08	.20-.40	1.0-1.2	.....	3.00-3.25
Stove plate.....	2.25-2.75	< .08	.60-.90	.60-.80		
Valves, large.....	1.25-1.75	< .09	.20-.40	.80-.10		
Valves, small.....	1.75-2.25	< .08	.30-.50	.60-.80	.....	low
Wheels, large.....	1.50-2.00	< .09	.30-.40	.60-.80		
Wheels, small.....	1.75-2.00	< .08	.40-.50	.50-.70		

\* Condensed from Porter's report *Trans. Am. Foundrymen's Assoc.*, Vol. 19, p. 134.

## PROPERTIES OF CAST IRON

**759. Shrinkage.**—At the moment of complete solidification gray cast iron expands, due to the precipitation of more or less graphite from the eutectic of austenite and cementite. Since graphite occupies more space than if chemically combined in the molten metal the total volume upon solidification is greater than that of the molten metal just before solidification. If phosphorus is high in the iron the initial expansion is soon augmented by the solidification of the phosphide eutectic which occurs at approximately 1000° C. As the temperature falls these expansions are gradually offset by contraction due to cooling. At 700° C., if the composition is suitable and the rate of cooling slow, another precipitation of graphite with consequent expansion takes place. With high silicon and high total carbon the latter expansion is very pronounced. The casting then shrinks continuously until it reaches room temperature. With proper regulation of the phosphorus, silicon and total carbon content, it is possible to avoid a coarse open-grained metal and still control shrinkage quite closely. Fig. 8 indicates the variations in shrinkage due to changes in silicon and size of casting.

In order to measure shrinkage, Keep has devised a sand mold with chills at each end for making a  $\frac{1}{2}$ -in. square bar 12 in. long.\* He determines the shrinkage by inserting a taper-scale between a chill and the adjacent end of the bar. By comparing the shrinkage of bars made in this mold with that of castings from the same metal it is possible to standardize the mixture which is best suited for a given purpose.

A simple shop test to determine sponginess or shrink cavities consists in molding a K-shaped casting. By breaking the branches of the K and noting the fracture, the soundness and density of the metal is revealed.

**760. Hardness of Cast Iron.**—From the machinist's point of view no property of cast iron is of more importance than its hardness, since the hardness determines the ease with which the iron can be filed or machined. We have seen that hardness increases with the proportion of combined carbon (Fig. 5) and is much influenced by the proportions of manganese and sulphur; that silicon up to about 3 per cent acts as a softener (Fig. 7) because it promotes the formation of graphite. Therefore, to secure an easily worked iron the proportion of combined carbon must be reduced to the lowest value consistent with requisite strength. In irons where high strength, closeness of grain, and ease of machining are properties much desired, the total carbon is kept low and silicon high.

For determining the hardness of cast iron, Keep has invented a drill

\* *Cast Iron*, p. 182.

test.\* In making this test a  $\frac{1}{8}$ -in. straight-fluted drill is pointed vertically upward and rotated at 200 r.p.m. The test-piece is held upon it with a pressure of 150 lb.; and an autographic record, showing the penetration of the drill in terms of the number of revolutions, is gotten. Variations in the slope of the traced curve indicate changes in hardness of metal, or dulling of the drill. The diagram is so set that a slope of  $0^\circ$  means that the rate of penetration is infinite while a slope of  $90^\circ$  indicates no penetration. With this device the hardness of white iron is  $90^\circ$  while machinery iron will vary between 25 and  $50^\circ$ , the value for a given iron depending upon the rapidity of cooling and the per cent silicon.

With the Brinell ball apparatus, which is perhaps the most satisfactory apparatus for measuring the resistance to indentation, the hardness number of machinery iron runs from 90 for very soft iron up to 200 for the dense, strong irons; the number for white iron ranges between 380 and 500.

**761. Influence of Composition and Rate of Cooling on Strength.**—We have already noted that the strength of cast iron is dependent upon the proportion of combined carbon, the proportion and form of the graphite, and upon the closeness and fineness of the grain of the iron.

From a consideration of these facts Sauveur advances the following argument. The maximum strength will be obtained when the matrix of the cast iron is approximately of eutectoid composition (i.e., pearlite), the graphite reduced to a minimum and the cooling done as rapidly as consistent with the attainment of the foregoing, in order that the graphite flakes may be small and the grain of the iron both close and fine. These conditions necessitate the use of mixtures having low total carbon and high silicon contents. The proportion of silicon should be sufficient to produce a eutectic of carbon in iron in order that the solidification period may be a minimum and the graphite finely divided. On the other hand, if the proportion of silicon is too high, the combined carbon content will be reduced below that required for a eutectoid matrix and the strength consequently impaired.

The above conception is, of course, modified by the influences of the other impurities present in the iron, by imperfections due to irregularities in molding, and by strains set up in cooling. Nevertheless it affords a rough basis for proportioning a mixture. Influences of other factors can best be ascertained by trial.

**762. Tensile Strength of Cast Iron.**—Although not commonly determined in the foundry, the tensile strength of cast iron is both an important

\* A similar apparatus has been perfected by A. Kessner. See *Jour. Iron and Steel Institute, Carnegie Fellowship Mem.*, Vol. 5, p. 10. A very small diamond drill has also been devised for such tests; see Jaggar drill described by H. C. Boynton, *Jour. Iron and Steel Inst.*, Vol. 70, p. 291.

property and a valuable index of the quality of the iron. In making the test it is quite necessary to avoid eccentric or oblique loading; therefore, a specimen provided with threaded ends like Fig. 3c; Chapter III, will furnish more precise results than the cheaper type of test-piece shown in Fig. 3f, Chapter III. The size of bar from which the test-piece is taken will exert a marked influence upon its strength, bars from large sections in most cases giving lower strengths than those from smaller sections of the same metal.

The tenacity of gray cast iron ranges from 12,000 lb. per square inch for soft, coarse-grained irons to 35,000 lb. per square inch for the hard, close-grained irons. Occasional reports are found of low-carbon cast irons with strengths above 40,000 lb. per square inch. The American Commission on Metal for Cannon in 1856 reported a maximum value of 46,000 lb. per square inch, and a tenacity of 47,500 lb. per square inch has been reported by the Wassiac furnaces of New York.\* White iron such as used for making malleable cast iron has a much higher strength than gray iron, commonly varying between 40,000 and 50,000 lb. per square inch. In small sections the strength of white iron may reach 60,000 or 70,000 lb. per square inch.

TABLE 2.—TENSILE STRENGTHS AND COMPOSITIONS OF STRONG CAST IRONS. (PORTER)

Reported by	CARBON.			Si.	S.	P.	Mn.	Tensile Strength (lb./in. <sup>2</sup> )
	Total.	G.C.	C.C.					
G. Dillner.....	3.15	2.50	.65	.90	.....	...	.30	35,600
R. T. Cunningham.....	3.10	2.70	.40	1.91	.060	.49	.74	31,890
H. E. Diller.....	3.23	2.15	1.08	2.36	.064	.33	.24	31,560
H. E. Diller.....	2.95	2.44	.51	1.83	.100	.65	.55	36,860
F. J. Cook and G. Hailstone.	3.09	2.29	.90	1.31	.101	.91	.33	36,600
W. Hatfield.....	3.23	2.68	.55	1.96	low	low	low	33,376
W. Hatfield.....	3.25	2.75	.50	2.19	low	low	low	34,944
G. A. Blum.....	3.20	2.50	.70	1.25	0.70	.70	1.00	high
F. J. Cook.....	3.00	2.28	.72	1.31	.056	.66	.43	35,430
J. A. Murphy.....	....	....	....	1.66	.065	.70	.90	36,000
J. A. Murphy.....	....	....	....	1.60	.063	.72	.85	37,300
J. A. Murphy.....	....	....	....	1.70	.070	.70	.75	30,400
J. A. Murphy.....	....	....	....	1.70	.075	.60	.92	31,300
T. D. West.....	3.07	2.44	.63	.94	.050	.44	.31	31,350
T. D. West.....	2.50	1.40	1.10	1.00	.050	.30	.60	33,000
T. D. West.....	3.52	3.10	.42	1.53	.050	.29	.45	30,000
T. D. West.....	1.71	.96	.75	.98	.080	.43	.43	34,000
T. D. West.....	3.18	2.05	1.13	1.19	.055	.41	.42	37,000
T. D. West.....	3.00	1.62	1.38	.71	.058	.54	.39	30,000

\* Incl. of ( ) p. 373.

Table 2 shows the analyses and tensile strengths of a number of strong cast irons reported by various authorities. Another set of data on tenacity and comparative hardness of high grade irons made and tested at the Watertown Arsenal is given in Table 3.

**TABLE 3.—COMPOSITION AND STRENGTH OF HIGH-GRADE CAST IRONS MADE AT THE FOUNDRY AT THE U. S. ARSENAL AT WATERTOWN, MASS. TEST-SPECIMENS GROOVED. (Rep. 1894, p. 247)**

Kind of Furnace.	CARBON.		Manganese.	Silicon.	Sulphur.	Phosphorus.	Tensile Strength, Lb. per Sq.in.	Hardness.
	Graphitic.	Combined.						
Cupola.....	2.440	0.900	0.335	1.137	0.113	0.572	27,700	16.07
Cupola.....	2.391	0.960	0.342	1.081	0.134	0.505	27,990	15.20
Cupola.....	2.487	0.744	0.461	1.511	0.118	0.521	31,980	17.35
Cupola.....	3.558	0.604	0.451	1.212	0.125	0.655	32,400	
Cupola.....	2.279	0.366	0.353	1.024	0.118	0.496	34,450	
Air-furnace...	2.492	0.736	0.448	1.231	0.125	0.816	32,980	
Cupola.....	2.393	0.432	0.450	1.090	0.140	0.497	31,110	
Cupola.....	2.727	0.299	0.462	1.363	0.125	0.477	31,810	15.83
Air-furnace...	2.058	0.778	0.464	1.500	0.115	0.619	29,100	20.47
Cupola.....	2.255	0.731	0.458	1.257	0.114	0.491	30,760	18.09
Cupola.....	2.890	0.458	0.388	1.645	0.105	0.487	27,320	
Air-furnace...	2.538	0.979	0.348	1.316	0.130	0.642	26,480	15.67
Air-furnace...	2.770	0.256	0.470	2.444	0.110	0.587	28,010	
Air-furnace...	2.751	0.357	0.435	1.908	0.095	0.420	29,120	11.08
Air-furnace...	2.538	0.634	0.355	1.222	0.090	0.766	28,520	21.04
Air-furnace...	2.577	0.185	0.361	1.146	0.115	0.762	31,020	
Air-furnace...	2.116	0.640	0.450	1.419	0.125	0.678	31,140	17.44
Cupola.....	2.825	0.479	0.361	1.062	0.076	0.238	32,010	16.82
Air-furnace...	2.481	0.687	0.454	1.175	0.120	0.673	31,990	

**TABLE 4.—COMPOSITION OF CAST IRON HAVING A HIGH TENSILE STRENGTH. (TURNER)**

	Woolwich Experiments, 1858, Average.	Silicon Experiments, 1885.	Rosebank Irons, 1886.				Dumbarton Irons.		Warrac Iron.	Average.
Tensile strength. Pounds per sq.in.	}									
		35,000	40,700	38,200	37,200	36,700	37,000	34,000	41,200	37,500
	%	%	%	%	%	%	%	%	%	%
Graphitic carbon.....	2.59	1.62	.....	.....	.....	.....	2.90	2.60	2.31	
Combined carbon.....	.....	0.56	0.36	0.58	0.52	0.40	0.32	0.30	0.78	0.475
Silicon.....	1.42	1.96	1.29	1.50	1.13	1.33	1.34	1.63	1.31	1.434
Phosphorus.....	0.39	0.28	0.56	0.47	0.41	0.70	1.09	1.10	0.29	0.587
Sulphur.....	0.06	0.03	0.06	0.07	0.06	0.05	0.14	0.12	0.08	0.074
Manganese.....	0.58	0.60	1.00	1.00	1.33	0.65	1.38	1.29	1.51	1.037

Some test results on excellent cast irons reported by Turner are presented in Table 4.

In specifying strength for sections thicker or thinner than the test-piece by which the quality of the metal is to be gauged, allowance must be made for the effects of differences in cooling. In standard specifications this has been done. Castings have been grouped into three classes: (1) light—maximum thickness under  $\frac{1}{2}$  in.; (2) medium—thickness between  $\frac{1}{2}$  in. and 2 in.; (3) heavy—minimum thickness over 2 in. The following minimum strengths for test-pieces like Fig. 3c, Chapter III,

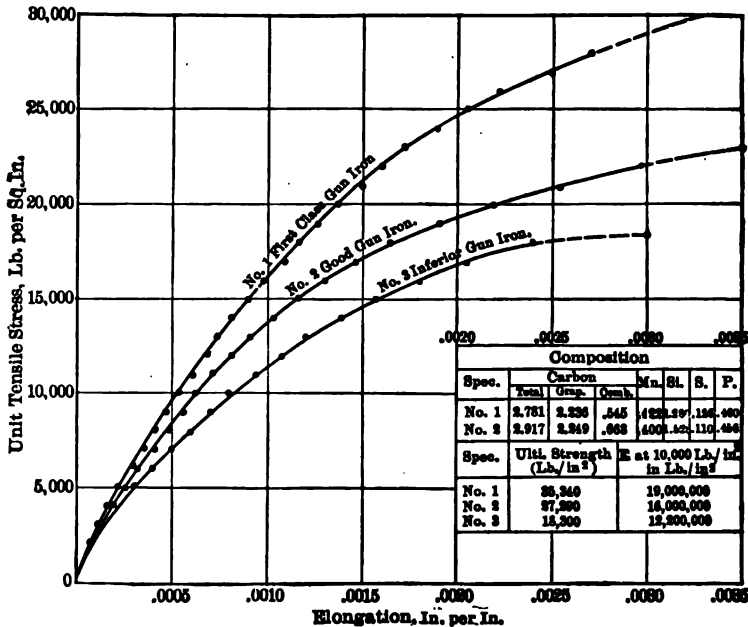


FIG. 10.—Stress-diagrams for Gray Cast Iron in Tension. (*Tests of Metals*, 1899.)

cut from  $1\frac{1}{4}$  in. round bars are standard: Light castings 18,000, medium castings 21,000, and heavy castings 24,000 lb. per square inch.

The general characteristics of stress-deformation curves for gray cast iron are shown in Fig. 10. As the curves show, the material has no well-defined elastic limit, but the apparent elastic limit determined as suggested in Art. 11 is found at approximately 60 per cent of the ultimate strength, which corresponds closely to the elastic ratio for structural steel and wrought iron. The stress-deformation diagram for white cast iron is a straight line. Inasmuch as the maximum unit deformation of cast iron is less than 0.005, it cannot be gotten from a tension test without the use of an accurate extensometer. Since measurements with the latter are



impractical in commercial work, no index of ductility or toughness is furnished by a tension test of cast iron.

**763. The crushing strength of cast iron is remarkably high, and it is this property which is considerably utilized in building constructions.** Ordinarily, owing to the necessity of using a testing machine of high capacity, tests of the crushing strength are not made. Since the crushing strength of small prisms is rarely less than five times the tensile strength, a rough estimate of the former may be gotten if the tenacity is known. When tests are made on small prisms with height at least one and one-half times the least lateral dimension, the crushing strength will range from

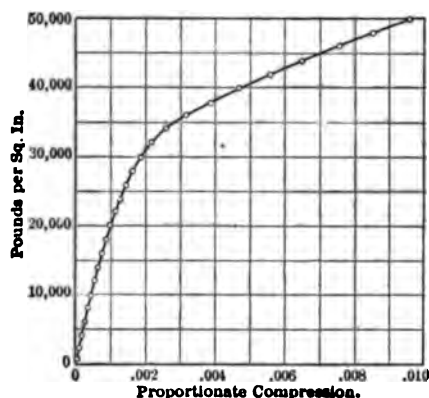


FIG. 11.—Average Results of Twenty-two Compression Tests on Gun Iron. (*Tests of Metals*, 1894.)

35,000 lb. per square inch for the soft open-grained gray irons to 200,000 lb. per square inch for the hard close-grained gray irons. (See also Fig. 7.) Machinery gray iron of good quality will have a crushing strength of 90,000 to 150,000 lb. per square inch. White cast iron is one of the strongest metals when subjected to compression, and often has a strength of 250,000 to 275,000 lb. per square inch. As in tensile tests, the position of the test-piece in the casting influences the compressive strength. Pieces cut from near the surface of a casting are

stronger than those from the interior. Specimens from small castings are stronger than those from large castings of the same metal. Sometimes variations in strength due to these causes amount to 100 per cent of the smallest values.

The stress-diagram of Fig. 11 is representative of good gray iron in compression, being the average of results from twenty-two tests of gun-iron. The specimens were 10.5 in. long and only 1 sq.in. in cross-section. Consequently they all failed by triple flexure, as columns, at an average stress of 63,000 lb. per square inch before the true crushing strength was reached. (The tensile strength of this iron averaged 33,500 lb. per square inch.)

In Table 5 are given the results of available tests on full-size cast iron columns. The very low strength of these columns is noteworthy and shows the necessity for testing full-size pieces, rather than small prisms, in order to gauge the strength of large members. The remarkable discrepancy here shown between the crushing strength of small specimens of cast

TABLE 5.—TESTS OF FULL-SIZE CAST-IRON COLUMNS

(Results of Tests made at Phoenixville, Pa., by New York Department of Buildings, 1897)

Number of Column.	Diameter, Inches.	Thickness of Metal, Inches.	Area in Square Inches.	Length in Inches. (l).	Actual Breaking Load per Square Inch in Pounds.	Radius of Gyration (r)	$\frac{l}{r}$	Breaking Load by Formula $p = 34000 - 88\frac{l}{r}$
I	15	1	43.98	190½	30,830	4.962	38.341	30,630
II	15	1½	49.03	190½	27,126	4.92	38.668	30,600
B2	15	1½	49.03	190½	24,434	4.92	38.668	30,600
B4	15½	1½	49.48	190½	25,182	4.965	38.318	30,630
(5)	15	1½½	50.91	190½	35,435*	4.936	38.543	30,610
(6)	15	1½½	51.52	190½	40,411	4.899	38.834	30,580
XVI	8	1	21.99	160	29,604	2.50	64.00	28,370
XVII	8	1½	22.87	160	28,229	2.486	64.361	28,340
(7)	6½	1½	17.64	120	25,805	1.786	67.189	28,090
(8)	6½	1½	17.37	120	26,205	1.805	66.483	28,150
G4	8	½	17.083	147½	25,969	2.741	53.903	29,260
F4	9	1	25.133	150	21,181	2.872	52.228	29,410
D4	12	1	34.588	162	30,810	3.906	41.475	30,350
C2	14	1	40.841	159½	25,401	4.609	34.661	30,950

\* Did not fail.

## RESULTS OF THE WATERTOWN ARSENAL TESTS

(Reports for 1887 and 1888)

Number of Column.	Least Diameter in Inches.	Approximate Thickness of Metal in Inches.	Least Area in Square Inches.	Length in Square Inches. (l)	Actual Breaking Load per Square Inch in Pounds.	Radius of Gyration (r).	$\frac{l}{r}$	Breaking Load by Formula $p = 34000 - 88\frac{l}{r}$
990	5.94	0.98	13.19	131.6	38,860	2.11	62	28,540
991	5.90	0.95	12.27	146.7	43,350	2.12	69	27,930
992	5.09	0.85	12.08	150.0	33,500	1.77	85	26,520
993	4.74	0.91	11.75	151.5	26,840	1.61	94	25,730
994	4.84	0.91	11.89	128.6	30,370	1.63	79	27,050
995	4.87	0.90	11.80	129.5	29,830	1.64	79	27,050
996	5.72	0.66	8.94	127.6	63,310	2.12	60	28,720
997	2.97	0.87	5.19	118.5	31,850	.97	122	23,270
998	3.00	0.88	5.27	118.7	29,990	.97	122	23,270
999	3.00	0.90	5.50	118.4	33,350	.97	123	23,180
1000	4.27	1.00	10.92	84.6	32,130	1.31	65	28,280
2000	8.66	1.36	31.10	157.0	(25,720)*	2.63	60	28,720
2001	7.87	1.31	26.33	156.9	(30,380)*	2.37	66	28,160
2002	7.17	1.16	21.75	156.9	25,470	2.16	73	27,540
2003	6.35	1.13	17.28	156.9	27,210	1.89	83	26,650
2004	5.57	0.77	13.22	156.4	25,100	1.71	97	25,360

\* Did not fail.

iron and that of full-size members is doubtless due to hidden defects such as shifted cores, blow-holes, and segregation. Nevertheless such defects are always likely to be present. As the computations in the table indicate, the straight-line formula,  $\frac{P}{A} = 34,000 - 88 \frac{l}{r}$ , very closely fits the observed results. In view of these data one is not warranted in using the commonly advocated formulæ which permit much higher loads than given by the above straight-line formula. Furthermore, the results emphasize the importance of carefully calipering cast-iron columns and pipes in order that variations in wall thickness due to shifting of cores may be detected.

**764. The Transverse Strength of Cast Iron.**—Transverse tests are preferred to other mechanical tests by foundrymen. The test has found much favor because the specimens are quickly and inexpensively prepared; the required testing machine is simple and readily operated by inexperienced men, and the results measure both strength and toughness. Tests are commonly made on the standard "arbitration bar," which is a cylinder  $1\frac{1}{4}$  in. in diameter and 15 in. long. Strength is measured by the center load which this bar will carry on a 12-in. span. An approximate measure of toughness is found by roughly estimating the energy of rupture from half of the product of the maximum load times the corresponding deflection.

The modulus of rupture of cast iron is not ordinarily computed in the foundry, but it forms the only satisfactory means of comparing bars of the same shape which differ slightly in size. Comparisons cannot well be made between bars differing considerably in size or in shape, since the rate of cooling causes radical changes in the grain structure of the interior. Variations in the moduli of rupture of square bars due to differences in area and in silicon content are well shown in Fig. 9. Square bars, in general, exhibit a slightly higher modulus of rupture than round bars of equal area. In a series of 152 tests by a committee of the A.S.M.E.\* in which bars  $\frac{1}{2}$  in. and 1 in. square and round bars of equal area were broken, the modulus of rupture of the square bars averaged about 5 per cent higher than that of the round bars. Variations in the modulus of rupture due to differences in shape of cross-sections are illustrated in Table 1, Chapter III. Machined bars are generally weaker than unmachined specimens. Tumbling in a rattler materially improves the strength and increases the hardness of the skin. Bars cast horizontally are strongest when the load is applied against the cope face.

The modulus of rupture varies from one and one-half to two and a quarter times the tensile strength in solid rectangular sections. (For the causes of this discrepancy, see Art. 27.) A comparison of the moduli of

\* *Trans. Am. Soc. M.E.*, Vol. 16, p. 542, 1066; Vol. 17, p. 675.

rupture and tensile strengths of round bars of machinery iron is given in Table 6.\*

TABLE 6.—A COMPARISON OF THE RESULTS OF TRANSVERSE AND TENSILE TESTS ON GRAY CAST IRON. (MATHEWS)

Each result represents nine tests. The standard threaded-end tensile specimens were turned from portions of bars tested transversely

Per Cent Si.	Span, Inches.	Mean Center Load, Pounds.	MODULUS OF RUPTURE.		TENSILE STRENGTH.		$\frac{S_m}{S_t}$
			Mean Value Lb./in. <sup>2</sup> ( $S_m$ )	Maximum Per Cent Variation from Mean.	Mean Value Lb./in. <sup>2</sup> ( $S_t$ )	Maximum Per Cent Variation from Mean.	
1.5	12	3000	47,100	15.9	25,600	5.2	1.84
	18	1935	45,600	10.7			1.78
	24	1425	44,700	11.2			1.75
2.0	12	2900	45,500	13.4	24,370	3.2	1.87
	18	1835	43,200	9.5			1.78
	24	1265	39,700	7.6			1.63
2.5	12	2880	45,200	6.4	24,660	7.5	1.83
	18	1905	44,900	16.7			1.82
	24	1400	44,000	17.5			1.78

It will be noted that the modulus of rupture for these irons averages about 1.8 times the tensile strength; furthermore, that the maximum variation in the results is much greater for the transverse tests than for the tensile tests. For Mathews' tests the tensile strength is approximately 8.5, 13, and 18 times the average center loads on the 12, 18 and 24-in. spans, respectively.

The effect of variation in span on the modulus of rupture is shown in Fig. 15, Chapter III. Fig. 12 shows that the energy of rupture per cubic inch decreases as the span is increased. Here the energy of rupture was approximated by taking half the product of the maximum load times the corresponding deflection and dividing by the volume of the specimen between the supports. Thick sections will, in general, have lower energies of rupture per unit volume than thin ones.

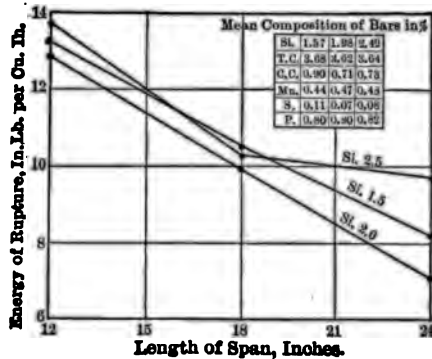


FIG. 12.—Influence of Length of Span on the Energy of Rupture of  $1\frac{1}{2}$ -in. Round Bars of Cast Iron. (Mathews.)

\* Compiled from Mathews' tests, see *Proc. A.S.T.M.*, Vol. 10, p. 299.

In Fig. 13 are shown autographic stress-diagrams of four kinds of cast iron. The tests were made on bars  $\frac{1}{2}$  in. square and 12 in. long. They all withstood a load of 450 lb. at the center, which gives a modulus of rupture of 64,800 lb. per square inch. Owing to the great differences in ultimate deflection, however, their resistances to shock vary greatly. By computing the areas under the load-deflection curves and dividing

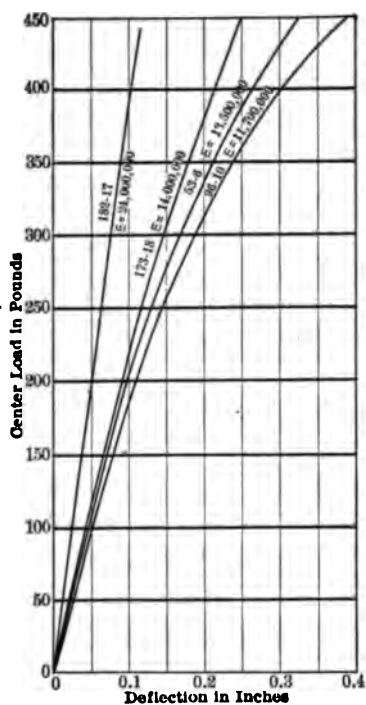


FIG. 13.—Cross-bending Autographic Stress Diagrams of Four Kinds of Cast Iron all of Equal Strength. (Keep, *Trans. A. S. M. E.*, Vol. 17, p. 677.)

by the volume of the test-piece between supports, we find their resistances are 10.0, 21.5, 28.9 and 35.1 in.lb. per cubic inch. For these four cases, the greatest error which would arise, due to approximating the energy of rupture by the method previously outlined, is 16 per cent.

In the form of the standard arbitration bar, gray iron should have a maximum deflection of at least 0.1 in. The minimum center loads should be 2500, 2900 and 3300 lb. for light, medium and heavy castings, respectively. These requirements, which are standard \* in this country, are equivalent to a demand for energies of rupture of 8.5, 9.8 and 11.2 in.lb. per cubic inch for these grades of metal. Good gray iron specimens of round or square section under  $1\frac{1}{2}$  sq.in. in area should have a modulus of rupture of at least 45,000 lb. per square inch.

**765. The modulus of elasticity of cast iron** varies quite as much as its strength. In this respect it differs markedly from all rolled iron or steel which, although varying greatly in ultimate strength, shows little variation in stiffness. From

the curves of Fig. 7 it appears that the stiffness and strength vary in the same way. Inasmuch as the stress-diagrams for cast iron are not straight lines the modulus may be calculated by dividing the working unit stress by the corresponding unit deformation. (See Art. 517.) If 10,000 lb. per square inch be the stress used to enter such diagrams, the modulus of elasticity for good gray cast iron will generally vary between 12,000,000 and 15,000,000 lb. per square inch. Strong irons occasionally have moduli of 20,000,000 lb. per square inch. The

\* See Specifications of the A.S.T.M. and Am. Foundrymen's Assoc.

modulus for white cast iron is considerably higher, commonly 20,000,000 to 25,000,000 lb. per square inch. Cast iron appears to have the same modulus of elasticity in tension, compression and cross-bending.

**766. Shock Resistance of Cast Iron.**—With the exception of tests on car wheels, few impact tests of cast iron are made. The resistance to shock is generally calculated from the energy of rupture in the cross-bending test. Where castings are likely to receive hard usage the impact test is valuable, especially if phosphorus or sulphur run high in the iron. Good quality of gray iron in standard arbitration specimens will exhibit an energy of rupture of 12 to 20 in.-lb. per cubic inch.

**767. Strength of Cast Iron in Shear and Torsion.**—Occasionally thin castings are punched, and therefore, a knowledge of the shearing strength of cast iron is valuable. Unfortunately there is little data on this property. G. Fremont records \* 267 tests in which he compared the transverse bending strengths ( $S_m$ ) with the shearing strengths ( $S_s$ ) for a wide variety of machinery cast irons. His bending tests were made on rectangular prisms 0.32 in. thick, 0.4 in. wide and supported over a 1.2 in. span; and the shear tests were made on fragments of the prisms used in the bending tests. Fremont's report contains values of  $S_s$  varying from 11,400 to 32,700 lb. per square inch and he states that good machinery iron should have a shearing strength above 25,600 lb. per square inch. From the diagram in his report the mean ratio of  $S_s$  to  $S_m$  is 1 to 2.6.

Fremont believes that the shear test of cast iron is very useful and has devised a very unique form of test. He secures a cylindrical specimen 0.3 in. in diameter from the casting by a core drill. He then fashions the cylinder into a square test bar 0.2 in. on a side and, beginning at the end of the bar nearest the surface of the casting, he shears off pieces at intervals of 0.12 in. By this method he readily secures a bar which truly represents the casting and also determines the variation in strength of the casting. Since the diameter of the hole bored in the casting is only 0.56 in., the structural damage to the casting due to sampling would in many cases be negligible.

A few tests on the shearing strengths of three grades of machinery iron are summarized in the table below. The shear specimens were approximately  $\frac{3}{8}$  in. thick and between  $\frac{1}{4}$  and  $\frac{3}{4}$  in. in width. They were tested in double shear in the Johnson shear tool, Art. 56. Each result represents two tests.

Kind of Cast Iron.	Brinell Hardness No.	Shearing Strength ( $\text{Lb./in.}^2$ )
(1) Soft and coarse-grained.....	92	20,500
(2) Fine-grained.....	150	33,700
(3) Rather hard, fine-grained.....	217	39,900

\* *Proc. A.I.T.M.*, VIth Congress, VI.

Under torsion cast iron fails in shear, and in tension, the plane of failure making an angle of about 45 degrees with the axis of the bar. Round bars of a good quality of gray cast iron  $1\frac{1}{2}$  in. to  $1\frac{3}{4}$  in. in diameter should exhibit a computed twisting strength  $S_s$  of 35,000 to 45,000 lb. per square inch. Three bars of a good grade of malleable iron  $1\frac{1}{2}$  in. in diameter tested by J. B. Kinnear\* had a tensile strength of 29,000 lb. per square inch and a computed twisting strength of 33,800 lb. per square inch. (Computed by formula for Art. 26.) The modulus of elasticity in torsion for the three bars varied from 949,000 to 8,220,000 lb. per square inch.

From four tests on hollow cylinders 1.60 in. in outer diameter and 0.05 in. thick made of iron No. 2 see above table, the computed twisting strength averaged 31,500 lb. per square inch, and the tensile strength 25,000 lb. per square inch. Since these cylinders were very thin, the com-

puted twisting strength was practically the same as the actual unit stress on the outside fibers. Therefore, for this iron the actual shearing strength in torsion was about 93 per cent of the transverse shearing strength and 126 per cent of the tensile strength.

**768. Shrinkage Stresses.**—The shrinkage of cast iron after it crystallizes is so great that, if not provided for, it causes excessive deformations which may develop very great stresses, even to rupture. The heavier or the thicker the casting the greater are these shrinkage stresses.

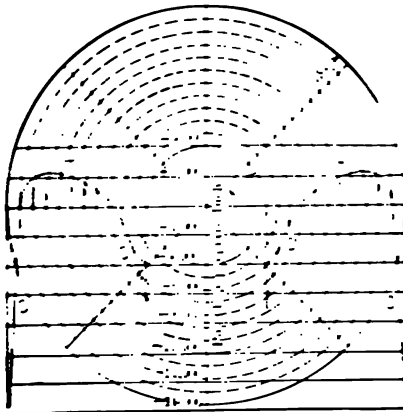


FIG. 14.—Shrinkage Stresses in Cast Iron Cannon 11 in. thick. *Tests of Metals.*

These have been studied in the case of cast-iron guns, and one such analysis is shown in Fig. 14. Here the metal was over 11 in. thick. The outer and inner surfaces cooled first, and the subsequent shrinkage of the interior put these parts in compression. But since the total internal stress across any diametral section must be zero, there being no external force acting, it follows that the total tensile stress must equal the total compressive stress. These were all found directly by cutting off a zone included between two transverse sections, and by cutting this up into a series of concentric rings as shown by the dashed lines in Fig. 14. Before cutting these, four diameters of each ring were carefully measured, and these same diameters were again measured after cutting out. An increase in mean diameter indicated

\* See paper on torsional strengths of various sections of cast iron in *Am. Machinist*. Vol. 40, p. 911.

an initial compression, and *vice versa*, the initial stresses being found from the equation

$$S = \epsilon E,$$

where  $S$  = unit stress in pounds per square inch;

$\epsilon$  = proportionate change in circumference;

$E$  = modulus of elasticity of the material.

In this way the stress-diagrams shown in Fig. 14 were computed and drawn by J. B. Johnson from the data furnished in the original report. It indicates that the interior surface was under an initial compressive stress of some 7000 lb. per square inch, the outer surface of some 13,500 lb. per square inch; while the interior was under a tensile stress of some 2000 lb. per square inch. Evidently the tension and compression areas on these diagrams must equal each other. This is a very simple illustration of such shrinkage stresses, because of its simple and symmetrical form. In complex forms it would be impossible to study or predict the character of these stresses. They are evidently less when all parts are made of approximately the same thickness.

**769. Strength of Cast Iron Increased by Shocks.**—Mr. A. E. Outerbridge has shown \* that castings which have been subjected to a great number of shocks or blows are from 10 to 15 per cent stronger under a static load and over 20 per cent stronger under impact than they are before receiving such treatment. He attributes this result to a sort of molecular rearrangement by which the cooling stresses are relieved. In other words, such treatment is equivalent to an annealing process. However, Keep has shown that this increase in strength is due to a densification of the surface produced by a smoothing and peening action of hammer or tumbling barrel.†

**770. Seasoning Cast Iron.**—If castings are allowed to age for several months before machining, the initial stresses caused by unequal contraction in cooling may be somewhat relieved. Such relief is probably due to molecular readjustments. Advantage of the improvement due to seasoning is taken by concerns making castings which must be accurately shaped or planed to a true surface.‡

**771. Effect of Repeated Heating on Cast Iron.**—Outerbridge§ has shown that cast irons having high contents of graphite and silicon swell markedly in volume, weaken, and finally crack when repeatedly heated between 1400 and 1600° F. With white irons this phenomenon does not occur. However, the brittleness and extreme hardness of white iron make it of

\* *Trans. Am. Inst. Min. Engr.*, Vol. 26, p. 176.

† *Trans. Am. Soc. Mech. Engr.*, Vol. 19, p. 366.

‡ See paper by R. Moldenke before *Inst. of Min. Engr.*, Feb., 1917.

§ *Trans. Am. Inst. Min. Engr.*, Vol. 35, p. 223.



little value for heat resisting castings. When castings are likely to be severely heated, expansion troubles can be considerably reduced by using a fine-grained dense gray iron, low in graphite. If castings are continually exposed to such high temperatures, steel is a better material than cast iron.

### MALLEABLE CAST IRON

#### NATURE AND IMPORTANCE

**772. Nature.**—White cast iron of suitable composition can be rendered somewhat malleable and ductile, and very greatly toughened by surrounding it with a suitable packing material and annealing at a bright red heat for several days. The iron before annealing should have all of its carbon in combined form, but the silicon, sulphur, and manganese contents must be so adjusted that the annealing may be accomplished at temperatures just above the critical range (1300 to 1500° F.). By the heat treatment the combined carbon is transformed into a special type of graphitic carbon, called *temper carbon*. The temper carbon is made up of finer, more-rounded, and more-uniformly disseminated grains than the graphite of gray cast iron.

If the packing material is loose and the furnace gases strongly oxidizing, practically all of the carbon will be removed from the outer layer of iron, the percentage removed decreasing toward the center of the casting. In American practice the entire annealing period is only five or six days and the carbon removal is small. Examination of a fractured cross-section of malleable cast iron reveals a thin white shell of impure carbonless iron about  $\frac{1}{64}$  to  $\frac{1}{32}$  in. thick surrounding a black core in which the grains of temper carbon are imprisoned among crystals of iron (ferrite). The color of the core in the fractured casting gives rise to the name *black heart*. In Europe the annealing process is carried on at a higher temperature and for a considerably longer time. This results in the production of a much thicker shell of decarburized iron, and a greater reduction in total carbon than in American practice. The castings so produced have a *white heart*, and are coarser grained and somewhat less strong than black-heart castings. By providing chills it is possible to make much thicker castings with the black-heart process than with the white heart, the latter being best adapted to the production of sections under  $\frac{1}{2}$  in. in thickness.

In strength, malleable cast iron is considerably superior to gray cast iron but inferior to steel castings. It is very much tougher than gray iron and, when well made, compares favorably with cast steel. Owing to lower melting temperature white cast iron shrinks slightly less than steel and in the annealing process an expansion takes place, due to the separa-

tion of the temper carbon, which makes the net shrinkage of malleable cast iron about the same as gray cast iron. Malleable castings are in general somewhat smoother and freer from blowholes than steel castings, and also more resistant to corrosion.

**773. Importance of Malleable Cast Iron.**—On account of good strength, high toughness, and moderate cost, malleable cast iron is much used for a large variety of small castings. About one-half of the million tons of malleable cast iron annually produced in the United States is used by railroads on rolling stock for car couplers, journal boxes, brake wheels, levers, door fasteners, hinges, pipe hangers, etc. A large tonnage is used in making parts of agricultural machinery and implements, while most of the remainder is used for pipe fittings, stove plate, hardware, ornamental castings and cheap tools. About two hundred foundries in the United States produce several times as much malleable cast iron as is made in all the rest of the world.

#### THE MANUFACTURE OF MALLEABLE CAST IRON

**774. Melting the Charge.**—The cupola, the air-furnace and the open-hearth furnace are the types of furnaces most used for melting the charge. Baby Bessemer converters and crucibles are used to some extent in Europe and the electric furnace is also forcing its way into this field.

The principal use of the cupola in malleable iron works is for making small castings. The process is run in much the same way as in smelting gray cast iron excepting that a larger proportion of fuel is required in order that the white iron, which is less fluid than gray, may be very hot when poured. The composition of pig iron for such castings according to Moldenke should be about as follows: Silicon 1.00 to 1.50, manganese <0.60, phosphorus <0.225 and sulphur <0.05 per cent. Small amounts of pig iron and sprues are charged between thin layers of coke or anthracite coal. Malleable or steel scrap cannot be advantageously used in this type of furnace. Iron is run continuously from the tap hole into ladles and poured as rapidly as possible. The cupola is the cheapest in installation, in upkeep and in operation, and the quickest smelting process in general use. However, owing to contact between the fuel and the iron and lack of means of control, burnt metal is sometimes produced. Furthermore, the annealing temperature required for cupola iron is higher than for the air-furnace or open-hearth product.

The air-furnace is used most in this country for making malleable castings. It is less expensive to install and operate than the open-hearth, although requiring a slightly longer time for smelting. It may be built in a wide variety of sizes, and can be operated discontinuously without impairing the quality of the iron. One of the main difficulties in air-

furnace operation is the possibility of burning the thin portions of the bath and consequent production of weak metal. By proper use of the air-furnace it is possible to produce a very good grade of malleable iron with less skill than required to run an open-hearth furnace. The charge of metal for the air-furnace consists of pig iron, white-iron scrap, and malleable scrap. When necessary to reduce carbon to proper limits (2.25 to 3.00 per cent) a small proportion of steel or wrought iron scrap is added after the pig iron has melted. For heavy castings the silicon content in the pig iron is run lower than indicated for cupola iron in order to avoid mottling of the hard iron castings. Fig. 6 indicates ranges in carbon and silicon contents in hard iron for making different grades of malleable cast iron.

The open-hearth furnace for malleable iron is built upon the same principle as that used in steel making, but in smaller sizes. With the open-hearth furnace the charge of metal is similar to that used in the air furnace. The fuel, however, is usually producer gas and air, both of which are separately heated by passage through hot checker works before they enter the furnace. Natural gas and vaporized fuel oil are sometimes used instead of producer gas. Although the most costly process on account of high cost of installation, upkeep and skilled labor for operation, the open-hearth furnace provides well-controlled melting conditions and furnishes the best malleable cast iron.

**775. Molding and Casting.** - Because of the lack of fluidity in white cast iron and the rapidity with which it chills, patterns must be provided with large runners and sprues. This should be done in order that the metal may be rapidly poured, also in order that a good head of metal may be provided to keep the mold full during solidification. Owing to the high shrinkage of white cast iron about double the allowance ordinarily made for gray iron patterns must be made in patterns for malleable iron. Consequently more care must be taken about joining thin and heavy parts. Suitable chills are often required to cool heavy sections with sufficient rapidity to make the iron white and to avoid excessive shrinkage strains at junctions with thin parts.

The molds used for malleable cast iron are similar to those made for gray castings but, owing to the number of castings made from the same pattern, there is a better opportunity for the effective use of molding machines and core-making machines. Metal molds, on account of the chilling action which they exert, are also successfully used for malleable castings.

In casting, it is quite necessary that the white iron be poured at a temperature sufficiently high to render it fluid; yet, on account of danger of burning, the metal cannot be held too long in the furnace. The narrow range of pouring temperature makes it necessary, therefore, to have

the metal handled and cast very promptly when it has arrived at a white heat.

After the white castings have cooled they are shaken out of the sand and cleaned by the methods used for gray castings. They are then trimmed of sprues, ground smooth, where necessary, and sorted.

**776. The annealing of the white castings** is an exceedingly important operation in the production of good malleable castings. The hard castings are carefully packed in rectangular boxes, called *saggars*, which are 16 by 24 inches in plan and a foot high. Mill scale from wrought-iron squeezers and silicious slag are often mixed to form the packing material. Hematite and pulverized magnetic ore are also used for this purpose. Leasman and Storey's \* experiments show that the oxidizing character of the packing material exercises no influence upon the decarburization of the iron, but that such change is due to the penetration of carbon dioxide generated in the furnace. The permeability of the packing to this gas is therefore the important factor in determining the carbon content in the skin of the castings.

The *saggars* are stacked 4 deep in an annealing furnace, which, in form and in principle of operation resembles a rectangular down-draft brick kiln. Gas, coal or oil is used to slowly heat the annealing furnace until the temperature of the castings is above the critical range. Storey states the latter is between 700 and 775° C. The temperature is held just above this range from sixty to seventy-two hours and then the castings should be cooled very slowly until well below the critical temperature. For black-heart castings about five days are required for annealing.

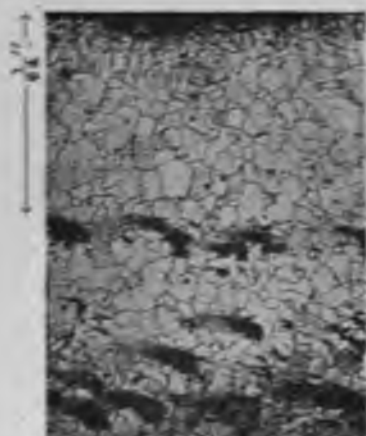
After removal from the *saggars*, the malleable castings are placed in a rattler, which is partly filled with bits of discarded malleable castings, and cleaned. They are then given whatever finishing is necessary. If crooked they are straightened but without heating.

## CONSTITUTION AND PROPERTIES OF MALLEABLE CAST IRON

**777. Composition and Constitution.**—Good black-heart malleable castings will generally contain about 2.0 to 2.8 per cent of temper carbon, 0.25 to 1.25 per cent of silicon, manganese under 0.30 per cent, sulphur below 0.5 per cent, and phosphorus under 0.25 per cent. The higher percentages of silicon are in most cases to be found only in small castings. For intricate patterns where fluidity of the molten metal is of great importance phosphorus may run to 0.3 per cent.

If the annealing is properly done no combined carbon will be found and the entire structure will consist of two main elements, ferrite and temper carbon, as shown in Fig. 15a. Such castings should possess maximum

\* See account of experiments at University of Wisconsin in *Foundry*, Vol. 42, p. 474.



(a) Good Malleable Iron. Note ferrite shell (top) free from temper carbon which appears as black patches farther in from the surface.



(b) Temper carbon in ferrite masses which are in turn surrounded by pearlite. (Storey.)



(c) Steely shell (top) caused by close compacting of packing material and lack of oxidation.

FIG. 15.—Photomicrographs of Good and Bad Malleable Iron ( $\times 70$ ).

ductility. If the annealing period is too short or the temperature too low, cementite will be present in addition to ferrite and temper carbon and the castings will be more brittle, although they may be stronger than the fully annealed iron. When the castings are cooled too rapidly after correct annealing, the structure shows the temper carbon embedded in ferrite which is in turn surrounded by pearlite, Fig. 15b. With too low annealing temperature followed by rapid cooling the structure consists of cementite particles embedded in pearlite masses with more or less temper carbon imprisoned in ferrite.\* Such iron is likely to be very brittle and non-uniform.

In order to determine whether the iron is of proper constitution a small rectangular lug is cast on the work. After annealing the lug is broken off with a hammer, the toughness of the iron judged by the energy required to remove the lug, and the fracture examined. Good black-heart iron will show a very thin white skin from  $\frac{1}{32}$  to  $\frac{1}{64}$  in. in thickness surrounding a bluish black or black core. The core should present a velvety appearance and be free from shrink cavities or white crystals.

#### 778. Testing of Malleable Cast Iron.—

In the foundry, the breaking of hard castings to determine whether the iron is mottled or white and the lug test of the malleablized iron are the principal mechanical tests. It is customary with more important work to run both tensile and transverse tests. For the former a bar shaped about like Fig. 3f, Chapter III, with a minimum diameter of  $\frac{1}{2}$  in. is specified. Recent specifications of the A.S.T.M. demand a tensile strength of 38,000 lb. per square inch and an elon-

\*Storey's Experiments.

gation of 5 per cent in 2 in. For the transverse tests rectangular bars  $\frac{1}{2}$ ,  $\frac{5}{8}$  and  $\frac{3}{4}$  in. deep, 1 in. wide, and 14 in. long are supported over a 12-in. span. These bars must withstand center loads of 900, 1400 and 2000 lb., respectively. The maximum deflections for the  $\frac{1}{2}$ ,  $\frac{5}{8}$  and  $\frac{3}{4}$  in. bars must not be less than  $1\frac{1}{4}$ , 1, and  $\frac{3}{4}$  in. respectively. Moduli of rupture corresponding to the above loads range from 64,800 to 64,000 lb. per square inch and the energies of rupture (by approximate method) from 93.6 to 83.3 in.-lb. per cubic inch.

A simple test for toughness and ductility, which seems to have found some favor in the foundry, is the curling test. This test is made on a wedge 6 in. long, 1 in. wide, and tapering from  $\frac{1}{2}$  in. to  $\frac{1}{16}$  in. in thickness, which has been cast and annealed with the given heat. The thin edge of the wedge is bent over with a hand hammer. It is then gripped rigidly at the thick end and held thin edge up under a drop hammer. The hammer is dropped on the specimen from a constant height giving blows of 70 ft.-lb. each. These cause the specimen to curl into a spiral.

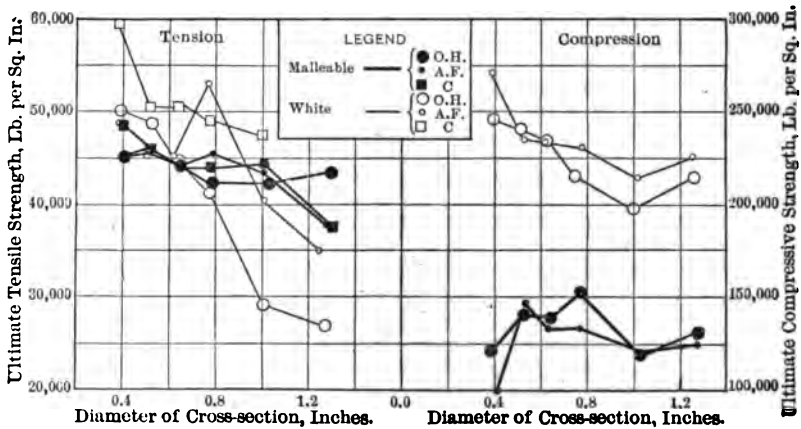


FIG. 16.—The Effect of Size of Specimen on Strength of Malleable and White Cast Irons. (Hathaway.)

Lack of ductility is signified by cracking of the wedge, toughness by the number of blows required to break it.\*

**779. Mechanical Properties of Malleable Cast Iron.**—Good black-heart malleable iron will have a tensile strength of 40,000 to 50,000 lb. per square inch with an elongation in 2 in. of 5 to 10 per cent. The per cent reduction in area is quite variable, usually being between 5 and 15 per cent for good iron. In compression, the strength of small prisms of good iron (load line parallel to skin) will range from 100,000 to 150,000 lb. per

\* Described by E. Touceda in *Foundry*, Vol. 43, p. 13.

square inch. Compression tests on struts \* where the ratio of  $l : d = 15 : 1$  show that such pieces buckle like wrought iron at an average unit stress of about 30,000 lb. per square inch. Fig. 16 shows the effect of variation in size of cross-section on the strength of round specimens for both white cast iron and malleable cast iron made by the air furnace, open hearth and cupola processes.† All specimens representing a given kind of furnace were taken from one melt. Each point on the diagram represents two or more tests.

TABLE 7.—MALLEABLE CAST IRON

CHEMICAL COMPOSITION UNANNEALED AND ANNEALED AND PHYSICAL PROPERTIES

Heat No.	Unannealed or Annealed	Total Carbon	Combined Carbon	Graphitic Carbon	Mn.	Si.	P.	S.	Cast Iron	TESTS OF ANNEALED SPECIMENS.				
										Maximum Strength lb. per Sq. in.	Per Cent Reduction.	Per Cent Elongation.	Number of Bars Averaged.	Hours Annealed.
391	Before annealing	3.92	2.96	0.06	0.18	0.69	0.13	0.00	0.07	55,100	5.5	5.2	3	108
	After "	2.95	1.52	80.0	19.0	69.0	140.0	0.04						
395	Before "	3.09	2.99	10.0	19.0	60.0	134.0	0.00	0.11	43,800	6.1	6.3	3	108
	After "	2.93	0.53	45.0	19.0	62.0	130.0	0.01						
400	Before "	3.70	2.96	13.0	18.0	75.0	142.0	0.05	0.53	44,900	7.7	10.3	3	108
	After "	2.56	0.31	25.0	19.0	74.0	142.0	0.01						
409	Before "	3.07	2.77	0.30	19.0	65.0	163.0	0.04	0.16	47,000	7.2	6.2	3	108
	After "	2.91	0.08	2.83	20.0	64.0	164.0	0.04						
422	Before "	3.26	2.65	0.61	17.0	69.0	156.0	0.01	0.31	45,300	9.1	8.2	3	108
	After "	2.95	0.36	2.59	18.0	61.0	151.0	0.06						
433	Before "	2.85	2.72	0.13	18.0	74.0	161.0	0.03	0.08	64,500	1.3	2.8	3	108
	After "	2.77	0.72	2.05	18.0	72.0	162.0	0.01						
436	Before "	2.88	2.73	0.15	18.0	90.0	196.0	0.00	0.30	38,900	4.3	6.2	3	108
	After "	2.58	0.51	2.07	18.0	87.0	192.0	0.00						
445	Before "	2.97	2.75	0.22	18.0	77.0	148.0	0.03	0.31	69,100	2.6	4.0	2	108
	After "	2.66	0.31	2.35	19.0	76.0	145.0	0.05						
451	Before "	3.08	2.85	0.23	18.0	96.0	123.0	0.03	0.93	45,400	7.2	8.0	2	108
	After "	2.15	0.08	2.07	19.0	96.0	129.0	0.00						
458	Before "	3.08	2.82	0.26	18.0	74.0	151.0	0.00	1.05	56,700	8.4	8.2	2	108
	After "	2.03	0.28	1.75	18.0	71.0	150.0	0.03						
459	Before "	3.03	2.81	0.22	19.0	70.0	195.0	0.00	0.97	44,200	2.1	4.2	3	108
	After "	2.06	0.27	1.79	20.0	70.0	192.0	0.03						
469	Before "	3.09	3.00	0.09	20.0	77.0	127.0	0.02	0.23	51,600	7.7	7.0	3	108
	After "	2.86	0.32	2.54	19.0	75.0	129.0	0.02						
171	Before "	3.08	2.92	0.16	23.0	70.0	154.0	0.02	0.41	46,600	9.8	8.5	3	108
	After "	2.67	0.09	2.58	22.0	70.0	156.0	0.02						
495	Before "	2.84	2.62	0.22	20.0	63.0	182.0	0.01	0.26	48,100	8.6	8.7	3	108
	After "	2.58	0.06	2.52	32.0	66.0	178.0	0.01						
510	Before "	3.26	3.23	0.03	40.0	64.0	136.0	0.04	0.14	46,000	5.9	5.3	3	108
	After "	3.12	0.53	2.59	40.0	67.0	135.0	0.03						
Av	Before annealing	3.04	2.85	0.19	21.0	73.0	154.0	0.05	0.38	49,810	6.22	6.0*	42	108
	After "	2.66	0.31	2.35	21.0	72.0	153.0	0.05						

\* C. H. Day in *Am. Machinist*, Vol. 29, pt. 1, p. 458; also see report by Miner and Blake in *Railway Age*, Vol. 31, p. 68.

† From a thesis by W. Hathaway, 1912, University of Wisconsin.

Table 7 shows results of tensile tests and analyses for a large number of tests by H. R. Stanford. In Stanford's tests the specimens were plain cylinders  $\frac{1\frac{1}{2}}$  in. in diameter while those used by Hathaway for tensile tests were proportioned like Fig. 3f, Chapter III. Stanford reports five tests on bars which were turned down until the cross-section had two-thirds of the area of the original piece, Fig. 17. From these tests it appears that the strength of the skin was twice as great as the tenacity of the interior. Tests by Hathaway on sixteen bars of malleable cast iron made by the air-furnace and a like number made by the open-hearth furnace indicate that black-heart castings as now made are approximately of the same tenacity, compressive strength and ductility throughout.

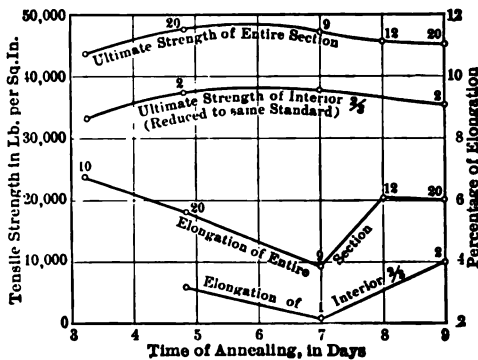


FIG. 17.

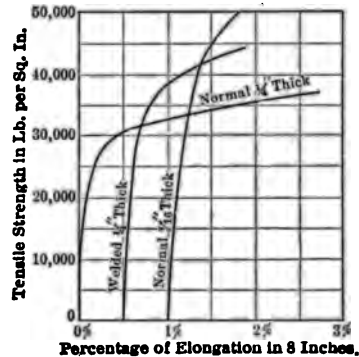


FIG. 18.

FIG. 17.—Tensile Strength and Elongation of Cylindrical Bars of Malleable Cast Iron  $\frac{1\frac{1}{2}}$  in. in Diameter. Numbers show how many tests were averaged for each plotted point. (Stanford.)

FIG. 18.—Tensile Tests of Malleable Cast Iron. Each curve represents two or three tests. (Berlin Testing Laboratory, 1886.)

Fig. 18 shows the results of tension tests on malleable cast iron of  $\frac{1}{4}$  in. and  $\frac{1}{16}$  in. thickness, and also of  $\frac{1}{2}$ -in. plates which had been welded together. The latter show a greater strength than the unwelded bars.

The modulus of rupture of first-class malleable cast iron in cross-bending will run from 65,000 to 90,000 lb. per square inch. The energy of rupture (by approximate method) will vary from 100 to 400 in.-lb. per cubic inch. Fig. 19 shows load-deflection curves for  $1\frac{1}{4}$ -in. round bars of white, malleable and gray cast irons. The white and malleable specimens from a given furnace are from the same melts. Each curve represents two tests, excepting for the gray cast iron which represents only one. The open-hearth malleable represents very good iron but the air-furnace malleable is of poor quality. The energy of rupture values in Fig. 19 represent the areas under the load-deflection curves; for the malle-



able specimens the tabulated values are about 50 per cent larger than the product of maximum load times corresponding deflection divided by twice

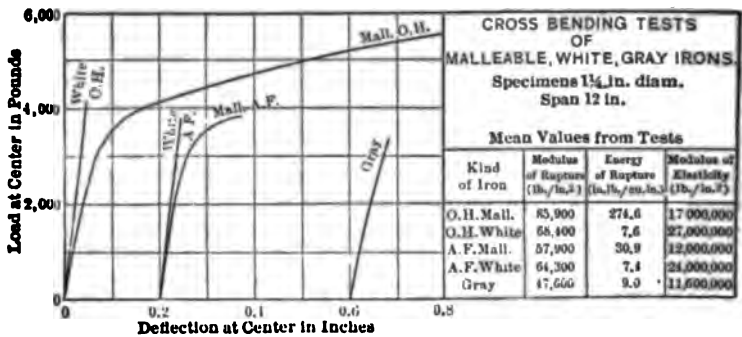


FIG. 19.—Typical Load-deflection Curves for White, Gray and Malleable Cast Irons.

the volume of the specimen between supports. The difference here shown between the energy of rupture of the malleable open-hearth iron and the

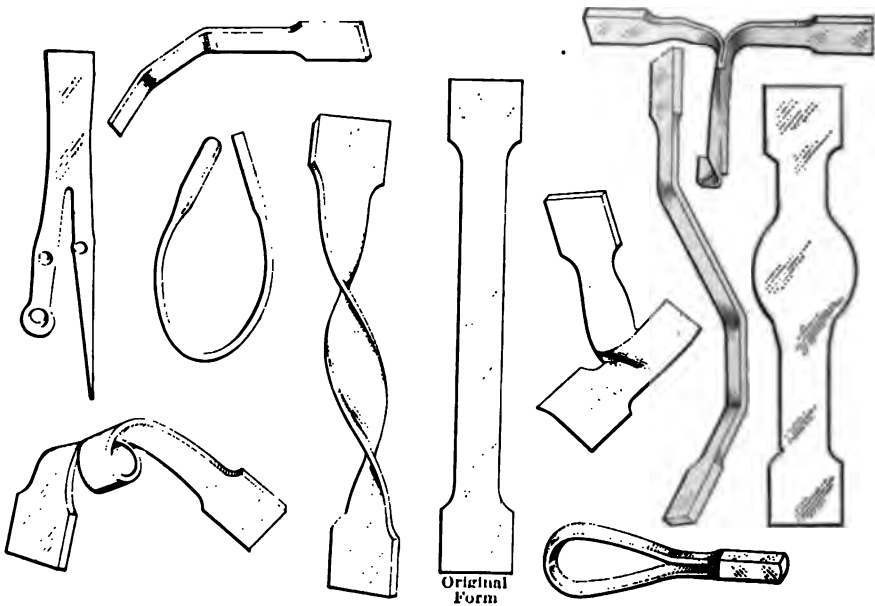


FIG. 20. —Examples of Cold-bending, Forging and Welding of Malleable Cast-iron Specimens, all being Originally like the Undeformed Bar in the Center. (*Berlin Testing Laboratory Communications*, Vol. 4, Pl. 3.)

corresponding white iron is representative of what may be expected with malleablizing.

For several years the students in the Materials Testing Laboratory at the University of Wisconsin have broken 1-in. round bars of air-furnace malleable cast iron over a 12-in. span in a Russell impact machine (Fig. 18, Chapter II). The majority of these specimens had energies of rupture in impact in excess of 350 in.-lb. per cubic inch and quite a number have withstood 500 in.-lb. per cubic inch without rupture.

The malleability and toughness of malleable cast iron made in Germany is well illustrated in Fig. 20. The plain bar represents the original shape from which all other forms were worked. In one case the ends of the bar were folded over and then welded together, while in another the metal forged like wrought iron. The remaining forms were deformed without heating.

## CHAPTER XXVI

### NON-FERROUS METALS AND ALLOYS \*

#### COPPER

**780. Production of Copper.**—Copper ores are among the most widely disseminated. Valuable deposits are found in nearly all countries of continental Europe, in Japan, Chile, Mexico, Canada, Spain, Peru, Australia and Africa. The United States, however, produces more copper than all other countries combined. The states which lead in the production of smelted copper are: Arizona, Montana, Michigan, Utah, Alaska, and Nevada.

In general, copper ores carry a much larger proportion of earthy material than the ores of iron and rarely contain more than 10 or 15 per cent of copper. The three principal groups of copper-bearing ores in order of importance are: The sulphides, native copper and the oxidized ores. Among the sulphides *chalcopyrite* ( $\text{CuFeS}_2$ , 34.5 per cent copper) and *chalcocite* or *copper glance* ( $\text{Cu}_2\text{S}$ , 79.8 per cent copper) are the chief minerals. Native copper is very extensively mined in northern Michigan; it is also found in New Mexico, Peru and China. In the Michigan deposits, native copper is found scattered through the lodes in particles of widely varying size. It constitutes up to 4 per cent of the ore mined and is generally very pure, although occasionally contaminated with arsenic; it is often called "Lake" copper. The oxidized ores are derivatives of the sulphides which have been broken down by the action of air and water. The more important are: *Cuprite*, the red oxide of copper ( $\text{Cu}_2\text{O}$ , 88.8 per cent copper), and the green carbonate, *malachite* ( $\text{CuCO}_3 + \text{Cu(OH)}_2$ , 57.3 per cent copper).

Nearly all copper is extracted by smelting; a small proportion is derived by wet methods in which the copper is withdrawn from the ore in the form of a sulphate or chloride. The oxidized ores are readily smelted in a special type of blast furnace using coke as fuel. Lake copper ores are first concentrated to form a mineral containing 70 per cent or more copper, which is then smelted in a reverberatory furnace. The sulphides, however, require a more complicated treatment, since it is not possible to reduce them directly to metallic copper. Smelting of the sulphide ores is commonly done as follows: The coarse lumpy ore is smelted in a blast furnace with or without previous roasting of a whole or part of the ore charged. The fine portions are usually roasted in

\* References: *The Metallurgy of the Non-Ferrous Metals* by W. Gowland, *Alloys and their Industrial Applications* by E. F. Law, *Metallic Alloys* by G. H. Gulliver.

reverberatory furnaces. These operations serve to concentrate the copper of the ore into a matte consisting principally of copper and iron sulphides with more or less of the sulphides of nickel, zinc, silver, and lead. Removal of the major portion of the iron and sulphur compounds is effected by placing the molten matte in a converter and oxidizing it with an air blast which enters just above the bath. After the smelting operation the crude copper ("blister copper") is cast into small pigs.

Refining of the crude copper may be accomplished by melting in a reverberatory furnace, or electrolytically. Refining in the reverberatory furnace is brought about by further oxidation of sulphides and by the cleansing action exerted by cuprous oxide on the base metals in the crude copper, the oxide being formed by air blown upon the molten bath. Since a large excess or a deficiency of cuprous oxide in the copper will make it weak and brittle it is necessary to remove any excess which remains after the impurities have been skimmed off. This is accomplished by additions of charcoal, and greenwood to the bath until the fracture of test ingots presents a flat salmon-red surface of silky texture. The copper is then at "tough pitch" and is ready for casting. Fire refining is used to give crude copper the malleability, ductility and toughness essential in plates, tubes and wires. It is also used to refine copper for alloys and to partially refine metal for anodes in the electrolytic process.

Electrolytic refining is used when an especially pure grade is wanted for electrical purposes, also when there is a considerable quantity of gold or silver associated with the crude copper. It is accomplished by passing a current through a copper sulphate solution from an anode consisting of crude copper, or partially refined copper, to a cathode of pure copper. By this method pure copper from the anode is plated upon the cathode and the precious metals settle to the bottom of the bath.

Copper for electrical purposes should contain less than 0.1 per cent of impurities (silver being counted as copper). Copper for castings generally carries less than 1 per cent of impurities.

**781. Properties of Copper.**—Besides possessing high resistance to atmospheric corrosion and high electrical conductivity, copper may be made very strong or very tough and malleable by suitable treatment.

The first and most general error to guard against in the matter of the strength of copper and its alloys is that of ignoring the mechanical treatment to which the

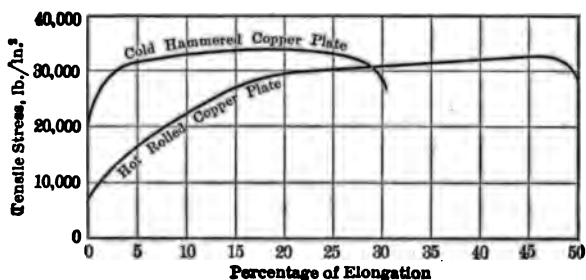


FIG. 1.—Typical Stress Diagrams for Copper Plate  $\frac{1}{4}$ -in. Thick. (Martens, *Berlin Testing Lab. Communications*, 1894.)

material has been subjected. Thus, in the case of copper plate, as shown by Fig. 1, a hot-rolled plate has an elastic limit of only some 7000 or 8000 lb. per sq. in., with an elongation of 50 per cent, while the same plate, cold-hammered, has an elastic limit of over 20,000 lb.

per sq. in., with an elongation of 30 per cent. Both have an ultimate strength of about 33,000 lb. per sq. in. When simply cast, without rolling or forging, both the elastic limit and the ultimate strength are much less, but copper is seldom used in this way.

The combined effects of mechanical and heat treatment on strength and ductility are well illustrated in the process of wire drawing. Thus the strength of hard-drawn copper wire will vary from 50,000 lb. per sq. in. for wire 0.5 in. in diameter to 70,000 lb. per sq. in. for wire 0.05 in. in diameter and the elongation will decrease from approximately 4 to 0.9 per cent. When annealed, wire of the above sizes will have a strength

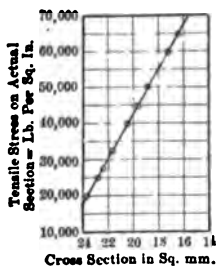


FIG. 2.

FIG. 2.—Showing a Linear Relation between Reduction of Area of Section and the Unit Stress on the Actual Section of Rolled Copper Plate  $\frac{1}{4}$  in. Thick. (*Rept. French Com.*, Vol. 3, Pl. 4.)

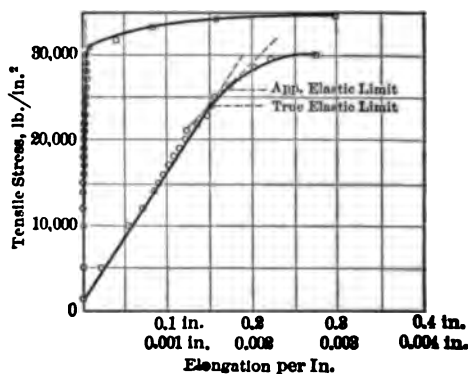


FIG. 3.

FIG. 3.—Typical Stress Diagram for Drawn Copper. (*Tests of Metals*, 1886, Vol. 2, p. 1673.)

of 35,000 to 40,000 lb. per sq. in. and an elongation of 35 to 25 per cent, being stronger and less ductile in the smaller sizes.

If the stress in a copper rod be computed for the actual cross-section at every stage of a tensile test, and the values so gotten be plotted against the diminishing cross-sectional area, the points will lie in a straight line, as shown in Fig. 2. This shows that the tenacity of copper increases regularly up to rupture with the reduction in area due to cold drawing.

The elastic limit of medium and hard-drawn copper wire lies between 50 and 60 per cent of its tensile strength. The metal does not, however, exhibit a well-defined yield point like steel, see Fig. 3. The modulus of elasticity of hard-drawn copper wire generally runs between 14,000,000 and 17,000,000 lb. per sq. in.

Hard-drawn copper may be annealed by heating to a temperature of 400° C. (752° F.). Rapid cooling does not interfere with softening of the

metal but repeated alternations of overstrain and annealing at the higher temperatures cause a marked growth in crystal structure. Effects on strength of wire of heating to various temperatures and quenching are shown in Fig. 4.

The electrical resistivity of copper having less than 0.1 per cent of non-metallic impurities lies between 0.155 and 0.159 ohm per meter gram at 20° C. The resistivity increases with the content of impurities and with amount of wire drawing.

**782. Uses of Copper.**—The major portion of the copper produced in this country is used for electrical purposes on account of its high conductivity. About one-fourth is used in making brasses and bronzes, and a smaller proportion is rolled into sheets for roofing and sheeting, into tubes for condensers and for other conductors which must withstand corrosion and possess fair strength and flexibility. Copper is also cast and beaten into various ornamental forms.

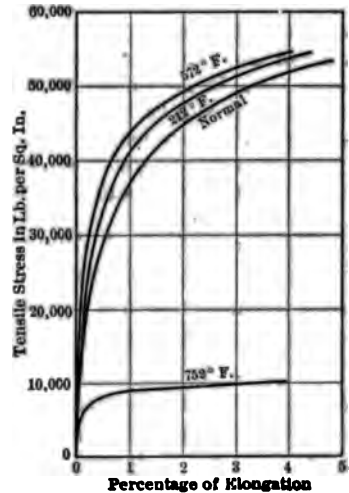


FIG. 4.—Effects of Heating Hard-drawn Copper Wires to Various Temperatures and then Quenching in Water. (Martens, *Berlin Testing Lab.*, 1894, Pl. I.)

## ZINC

**783. Production of Zinc.**—Like copper most of the world's supply of zinc is obtained from sulphide ores, *zinc blende* or *black jack* ( $\text{ZnS}$ , 67 per cent zinc), which ordinarily carry from one-third to one-half zinc. The ores are very often found associated with the sulphides of lead (*galena*), iron (*pyrites*), and copper. The principal sources of supply are the Rhine districts of Germany, Great Britain, Spain, Sweden and within the United States,—Missouri, Colorado, Wisconsin and Kansas. Zinc carbonate, *calamine* ( $\text{ZnCO}_3$ , 52 per cent zinc); the zinc silicates *Hemimorphite* and *Willemite*, and *Franklinite* (an ore of iron, manganese and zinc) form less important sources of supply. The carbonate is of chief importance in the Mediterranean countries while the silicates and *Franklinite* are the sources of an important supply in New Jersey.

The principal features in the extraction of zinc from its ores are the reduction of the sulphides and carbonates to the oxide form and the subsequent distillation of the oxide. The sulphide ores are finely ground and slowly roasted in reverberatory furnaces until nearly all of the sulphur is expelled. Carbonate ores and silicate ores are often calcined

in shaft furnaces before being distilled. After roasting or calcination, the ore is mixed with a nearly equal amount of finely ground coal and shoveled into fire-clay retorts. By carefully controlling the temperature of the retorts at a white heat, carbon monoxide is produced and the zinc, thus relieved of its oxygen, is collected and cooled to liquid form in condensers. From time to time molten zinc is tapped from the condensers, skimmed and poured into molds. The zinc so cast is called spelter. Most of the spelter made in the United States is sufficiently pure for industrial purposes. When contaminated with lead or iron, it is further refined by melting at as low temperature as possible. By so doing a separation of these metals is effected through the differences in their specific gravities. In 1915 the electrolytic refining of zinc on a commercial basis was successfully begun at several places.

**784. Properties of Zinc.**—The most important property of zinc is, without doubt, its resistance to atmospheric corrosion. However, in order to make use of it for protective purposes, as in fruit jar covers, cans and battery zincs where it must be deformed into the shape wanted, zinc must possess considerable ductility and strength. Like most metals the strength and ductility of zinc are much influenced by composition, heat treatment, and mechanical work. Observations show that lead tends to make spelter roll easier but it also softens, weakens and reduces ductility. Consequently it should not exceed 0.1 per cent in spelter used for making cartridge brass or condenser tubes. Iron and cadmium embrittle and harden zinc and are, therefore, a detriment in spelter which is to be rolled or used for galvanizing. For such purposes the content of these elements in the spelter should be very low, probably not over 0.02 or 0.03 per cent.

Data for estimating the strength of cast zinc are afforded by the tests of Rigg and Williams\* which appear in Table 1. They found that small differences in the casting temperature of the spelter and slight variations in the temperature of the cast-iron molds caused marked changes in the sizes of the crystals and in the tenacity of the specimens. For example, note the wide variation in the tenacity of the specimens of the first samples of Prime Western No. 1 and No. 2 spelters. The specimens having the maximum strength were much finer grained than those of minimum strength. Similar results were obtained in the transverse tests. In compression, however, the loads carried at a given deformation were far more uniform. The compression specimens also exhibited a high degree of plasticity which was wholly absent in the tensile specimens. Rigg and Williams attribute the high compressive strengths of the first samples of Prime Western No. 1 and Prime Western No. 2 to the high cadmium contents in these spelters.

The influence of the reduction in rolling on the tenacity of zinc was investigated by Prof. H. E. Moore.† He found, for example, that the ten-

\* *Proc. A. S. T. M.*, Vol. 13, Pl. 19.

† *Bulletin* No. 52, Engr. Expt. Sta. University of Ill.

sile strength of the zinc, which was 9060 lb. per sq. in. when cast, became approximately 22,000 lb. per sq. in. when rolled into plate 1 in. thick, and 25,000 when rolled into sheets 0.006 in. thick, both being pulled in the direction of the rolling. The strength across the grain (transverse to the rolling) was somewhat higher, especially for very thin sheets, than the strength in the direction of the rolling, but the ductility was pronouncedly lower. In the direction of the rolling the elongation in 8 in. varied from 4.85 to 21 per cent for specimens ranging in thickness from 1 in. to 0.006 in.

Zinc either cast or rolled shows no well-marked yield point. Moore found the limit of proportionality in tension varied from 2900 to 5800 lb. per sq. in. for the specimens he tested but nearly all of his specimens exhibited permanent set at much lower stresses.

TABLE 1.—THE STRENGTH OF CAST ZINC (Riggs and Williams)

Most of the values represent four tests, some only three. Tension test-pieces were like Fig. c' Ch. III, with 1-in. gage length. Transverse specimens were 1½-in. cylinders 15 in. long with 12-in. span. They were not machined. Compression specimens were 1 in. in diameter and 2.6 in. long. All test-pieces were gotten from 1½-in. cylindrical rods 17½ in. long cast on end in cold iron molds. In all cases the upper 2½ in. of the castings were discarded.

Grade of Spelter.	Sample No.	ANALYSIS, PER CENT.			MEAN STRENGTH, LB./IN. <sup>2</sup>			Maximum Deflection on 12-in Span.	MAXIMUM VARIATION FROM MEAN IN PER CENT.		
		Pb	Fe	Cd	Tension.	Compression at 20% De- formation.	Modulus of Rupture.		Tension.	Compression.	Mod. Rupt.
High grade.....	1	.041	.014	.000	6,262	24,230	11,630	0.25	35	3	5
High grade.....	2	.040	.016	.000	4,330	23,640	10,570	0.25	15	2	5
Intermediate.....	1	.194	.016	.000	4,340	23,030	10,160	0.30	25	5	5
Intermediate.....	2	.190	.017	.000	5,095	23,150	12,360	0.31	19	4	20
Brass special.....	1	.474	.013	.000	11,980	28,070	16,550	0.29	20	3	34
Brass special.....	2	.484	.031	.000	4,330	24,830	13,110	0.21	24	5	35
Prime Western No. 1....	1	.19	.032	.250	7,710	38,770	11,020	0.13	46	0.3	23
Prime Western No. 1....	2	.42	.087	.079	3,700	29,970	10,050	0.13	19	5	7
Prime Western No. 2....	1	.68	.010	.274	10,800	39,490	16,250	0.18	34	2	18
Prime Western No. 2....	2	.15	0.11	.046	4,670	28,450	10,370	0.19	22	6	9
Dross.....	2	2.09	3.51	.043	7,340	34,380*	15,300	0.05	13	11	10

\* Failed completely.

Moore found the average ultimate shearing strength of eleven specimens of rolled zinc in punching tests was 19,400 lb. per sq. in. and for twelve specimens in double shear 17,100 lb. per sq. in.

From a large number of tests by Moore, the modulus of elasticity of rolled zinc lies between 10,000,000 and 15,000,000 lb. per sq. in., averaging about 12,000,000 lb. per sq. in. Six tests on cast zinc in tension gave



11,025,000 lb. per sq. in. Four tests on cast zinc in compression gave an average modulus of only 6,900,000 lb. per sq. in.

**785. Uses of Zinc.**—As a protective coating zinc is much used on iron and steel plate, boiler tubes, fruit jar covers, cans for resisting corrosion. It is used in making shoe nails, zinc etchings and the negative pole pieces of batteries. As a constituent of brass, German silver and some of the bronzes, zinc is also much used.

## ALUMINUM

**786. Production of Aluminum.**—Aluminum is derived from bauxite of which the principal constituents are hydrated oxides of aluminum and iron with some silica. The important deposits of bauxite which are being worked are located in the United States and in France. Arkansas, Tennessee, Alabama and Georgia are the producing states. Between 30 and 35 per cent of aluminum is the proportion generally found in the ores which are being mined.

The first step in the process of extraction of aluminum is the production of its oxide alumina from bauxite. Alumina is obtained by roasting the coarsely granulated bauxite at a temperature sufficient to drive off the water from the iron oxide. The treated bauxite is then finely ground and heated under pressure for several hours with a solution of sodium hydrate. This solution is diluted and filtered and a little sodium hydrate added to the filtrate. It is then agitated for several hours to precipitate the hydrate which is separated, washed, and calcined at a temperature of approximately 1000° C. The process results in the production of alumina with perhaps 1 per cent of the oxides of sodium, iron and silicon as impurities.

Aluminum is extracted by electrolytic decomposition of alumina in a molten bath of cryolite (a fluoride of alumina and sodium). The cryolite is placed in a shallow rectangular hearth provided with a coke bottom which serves as the cathode, and several vertical carbon rods suspended in the bath, which serve as anodes for the electric current. After the cryolite is introduced into the furnace and melted by the passage of the electric current, the alumina is thrown onto the bath. As it melts it is dissociated into aluminum and oxygen, the former settling onto the cathode at the bottom of the bath while the oxygen goes to the anodes and, forming carbon monoxide, escapes from the bath. From time to time a portion of the aluminum is tapped from the furnace and alumina and cryolite are added to replenish the bath. The metallic aluminum thus obtained usually contains from 0.2 to 2 per cent of silicon and iron as impurities. "Pure," or No. 1, aluminum usually carries about  $\frac{1}{2}$  per cent of these impurities.

Refining of alumina is done principally in the United States, France, Norway, Switzerland, Canada and Austria. The production of aluminum is increasing very rapidly, especially in the United States and Norway. The entire production of the world is about 150,000 tons.

**787. Properties of Aluminum.**—Aluminum is a white metal of high metallic luster. It is harder than tin, having in rolled form a hardness of approximately 40 on Brinell's scale. Being second only to gold in malleability, it can be rolled into sheets 0.0005 to 0.0006 in. in thickness and may

be hammered into leaves much thinner. Aluminum is one of the lightest of the metals of construction and, in proportion to its weight, very strong. The specific gravity ranges from 2.56 for castings to 2.7 for the densest types of mechanically worked parts. It is less ductile than copper but excels zinc, tin, and lead in this respect. The results given in Table 2

TABLE 2.—TENSILE PROPERTIES OF ALUMINUM  
(Alloys Research Committee)

Composition:—Al = 99.53–99.54, Si = 0.24–0.31, Fe = 0.12–0.14 per cent.

Condition of Metal.	STRENGTH IN LB./IN. <sup>2</sup> AT		Per cent Elongation in 2 in.	Per cent Reduction in Area.
	Maximum Load.	Yield Point.		
Cast (cooled slowly) . . . . .	10,270	4,280	1.9	27.3
Cast (cooled quickly) . . . . .	11,310	5,380	22.0	33.8
Cast in chills . . . . .	11,700	5,150	37.0	60.7
Rolled to 1½-in. round bars . . . . .	14,560	9,850	35.5	79.3
Rolled to 1½-in. round bars . . . . .	16,130	14,570	30.5	81.2
Cold drawn to 1½-in. diameter . . . . .	19,500	19,050	19.5	73.3
Cold drawn to 1½-in. diameter and an- nealed . . . . .	16,620	15,690	23.0	82.0
Rolled into sheets ½-in. thick . . . . .	18,750	17,700	In 3 in. 10.6	
Rolled into sheets ½-in. thick and an- nealed . . . . .	13,240	6,050	41.0	
Rolled into sheets ⅞-in. thick . . . . .	20,290	19,270	6.3	
Rolled into sheets ⅞-in. thick and an- nealed . . . . .	13,220	6,720	31.0	
Rolled into sheets ⅞-in. thick . . . . .	20,450	18,820	3.7	
Rolled into sheets ⅞-in. thick and an- nealed . . . . .	13,060	7,170	36.3	

show the tensile properties of pure aluminum as determined by the Alloys Research Committee.\* When hard-drawn into very fine sheets or wire the ultimate tensile strength may reach 40,000 to 50,000 lb. per sq. in. The compressive strength of cast aluminum in cylindrical specimens with length twice the diameter is 12,000 per sq. in. and the elastic limit 3500 lb. per sq. in. according to reports of the Aluminum Company of America.† The modulus of elasticity of cast aluminum is approximately 9,000,000 lb. per sq. in.

The coefficient of electrical conductivity of aluminum is about 62, silver being 100. On account of its light weight, a bar of given length is

\* See Eighth Report, *Proc. Inst. Mech. Engrs.*, 1907, Pt. 1, p. 57.

† See *Properties of Aluminum*, published by the company.

twice as good a conductor as a copper bar of the same length and weight. However, owing to the superior resistance of copper to repeated stress and its lower coefficient of thermal expansion, it is generally preferred to aluminum for transmission purposes.

Aluminum castings contract about 0.2 in. per foot in cooling from the molten state and are quite brittle (hot short) when solidifying. Aluminum may be cast either in sand or chill molds. With sand molds great care must be exercised to use sand as dry as can be worked and to avoid hard ramming of molds and cores. In pouring, the temperature of the metal must be kept as low as possible and the rate of pouring must be very slow. Aluminum castings are quite open grained, consequently due allowance for porosity must be made in designing the thickness of cylinder walls and pressure tanks to be made of it.

Aluminum parts can be annealed by gradually heating to temperatures between 350 and 500° C. and allowing them to soak for a short time at the temperature which has been found suitable for the given class of work.

According to manufacturers statements,\* aluminum is highly resistant to the attack of nitric acid, is slowly dissolved by concentrated sulphuric acid and is soluble in hydrochloric acid. At ordinary temperatures sulphur, carbolic acid, salt water, vinegar, sea water, carbonic oxide, and sulphuretted hydrogen do not attack it; but it is rapidly corroded by the caustic alkalis.

**788. Uses of Aluminum.**—On account of the softness and porosity of aluminum castings and the superiority of its alloys with copper or zinc, little aluminum is used in cast form. However, it can be worked at much lower temperatures than iron or copper; and, inasmuch as its density and mechanical properties are greatly improved by rolling, heating, drawing, extruding or drop-forging, it is much used after such treatment. Very light tubing and wire are drawn from extruded sections of aluminum. It is rolled into sheets which in turn, may be stamped into a variety of shapes, in many cases without annealing.

Aside from the uses mentioned, aluminum is employed for portions of automobile bodies, for cylinders and pistons in automobile and aeroplane engines, for tips on bullets, for rivets, and kitchen utensils.

#### LEAD, TIN AND NICKEL

**789. Lead.**—Nearly all lead is derived from ores containing lead sulphide, *galena* ( $\text{PbS}$ , 86.6 per cent lead). Lead carbonate, *cerussite* ( $\text{PbCO}_3$ ), and lead sulphate, *anglesite* ( $\text{PbSO}_4$ ), are formed by decomposition of *galena* and sometimes constitute the outcropping portions of the *galena* ore deposits. Lead ores are comparatively lean, averaging in the United

\* See *Properties of Aluminum*.

States about 5 to 6 per cent of lead. The United States, Spain, Germany, Mexico and Australia normally produce over four-fifths of the world's supply of lead. Within the United States the chief sources of lead are Missouri, Idaho, Utah and Colorado.

Lead is extracted from its sulphide ores, generally in two steps. The preliminary operation consists in reducing the sulphur content by roasting the raw ore in pots or sintering it in shallow pallets. It is then smelted in a blast furnace. The products of the blast furnace are: lead bullion, containing more or less antimony, arsenic, copper, iron, silver and gold as impurities; a matte carrying copper and the remainder of the lead; sometimes a speise containing arsenides of iron and traces of the precious metals; and slag. If the bullion carries much of the impurities or the precious metals it is refined in reverberatory furnaces. The matte and speise are further treated to remove the lead which they carry.

Lead has a blue-gray color and exhibits a dull metallic luster when freshly fractured. Exposure to moist air causes oxidation and loss of luster. Lead is so soft that it may be scratched with the finger nail and so malleable that it can be readily rolled into very thin sheets as in thin foil. It lacks ductility, however, and cannot, therefore, be drawn into fine wire. Lead is formed into pipes with hydraulic presses which extrude the hot metal through dies. Both the softness and the specific gravity of lead are reduced by the presence of the common impurities antimony, arsenic, zinc and copper. Magnesia, however, has a still more powerful hardening influence, only 2 per cent being required to raise the Brinell hardness of pure lead from about 6 to 20. Pure lead pigs have a dull, dead sound when struck with the hammer, but the presence of impurities increases resonance. Lead castings shrink about  $\frac{1}{8}$  in. per foot in cooling.

The tenacity of cast lead generally lies between 1500 and 2000 lb. per sq. in.; in hard lead wire the strength may reach 3000 lb. per sq. in.

For the manufacture of sheets, pipes, solder, pewter and white lead a pure grade of lead containing less than 0.1 per cent impurities is wanted. For bearing metals and some of the alloys, hard lead which carries from 15 to 20 per cent of antimony is often used.

**790. Tin** is obtained in the Federated Malay States, Bolivia, Dutch East Indies, Siam, England, Australia and Africa from the black oxide of tin, cassiterite ( $\text{SnO}_2$ , 78.6 per cent tin). The total output of tin is about 112,000 tons annually. The principal deposits of the ore are found in alluvial sands, whence the name stream tin, and in veins or lodes, called lode tin. Stream tin deposits commonly contain less than 1 per cent, whereas the lode ore carries about two-thirds tin.

Stream tin ores are concentrated by washing and roasting processes and lode ores are crushed. Ores containing large amounts of sulphur or arsenic are roasted to oxidize these impurities. After these preliminary treatments the dressed or roasted ore is smelted at a high temperature ( $1000^\circ \text{C.}$ ) in a reverberatory furnace, or, if the ores

are very pure, a blast furnace is sometimes used. The crude tin thus produced carries more or less iron, copper, lead, arsenic, antimony, and tungsten. Since many of the alloys of tin and its impurities have higher melting-points than the pure metal, the latter can be separated by raising the temperature of the crude tin just above the melting-point of pure tin. Further refining is brought about by aerating the molten tin through violent agitation, thus producing more complete oxidation of the impurities. This is accomplished by submerging logs of greenwood in the bath of molten tin (poling) or by repeatedly pouring the molten tin from ladles (tossing).

Considerable quantities of tin are now recovered by electrolytic and chemical methods from scrap tin plate.

The best grades of Asiatic tin carry less than 0.1 per cent of impurities; but the lower grades from China and Bolivia, and tin recovered from scrap, may contain from 1 to 5 per cent of impurities,—lead, antimony and copper being the principal associated metals.

Tin is a silvery-white, lustrous, and extremely malleable metal as is evidenced by its form in tin-foil. Its specific gravity is 7.3 and it melts at 232° C. but does not volatilize until the temperature is raised above 1200° C. Tin is harder, more ductile and somewhat stronger than lead. The presence of iron, copper, or lead renders tin harder and more brittle, whereas small percentages of arsenic and antimony reduce its strength and hardness. The ductility of cast tin is influenced by the casting temperature, too high or too low temperature causing brittleness. Reported values of the tensile strength of tin vary from 2500 to 5000 lb. per sq. in. In ductility it equals soft steel. Tin is somewhat stronger in compression than in tension.

On account of the resistance of pure tin to corrosion it is much used in sheet form for roofing, for coating cans, and as a coating on soft steel sheets. Considerable tin is used in making the bronzes and other alloys and a small proportion is made into tin-foil.

**791. Nickel** is gotten almost entirely from two sources, the nickeliferous magnetic pyrites found in the Sudbury District of Ontario, Canada, and to some extent from the hydrated nickel-magnesium silicate of New Caledonia, an island east of Australia. The pyrite ore usually contains about 3 per cent nickel, 2 per cent copper, with iron and sulphur constituting the major portion of the residue. The silicate ores generally carry from 6 to 8 per cent of nickel. About 50,000 tons is the world's yearly production of metallic nickel.

In order to extract nickel from the sulphide ores, it is first necessary to roast the ore to reduce the sulphur content. The roasted ore is then smelted in a blast furnace and a crude matte of nickel, iron and copper is formed. By Bessemerizing this matte the iron is removed, leaving a more pure matte of copper and nickel sulphides. Nickel with 1 to 2 per cent of impurities may be obtained from the Bessemerized matte by smelting in a reverberatory furnace with coke and sodium sulphate. By this process the copper and iron are formed into a matte of lower specific gravity than nickel sulphide. The nickel sulphide is withdrawn from the bottom of the molten bath and remelted until

the desired purity has been obtained. It is then roasted to form nickel oxide which can be reduced to metallic nickel by smelting with charcoal in iron tubes.

The Bessemerized matte may be more completely purified by the Mond process. In the latter process the matte is first crushed, ground and roasted. The oxides are then treated with dilute sulphuric acid to remove the major portion of the copper. The residue is partially reduced by hot producer gas and volatilized at a lower temperature into nickel carbonyl. By passing the latter through a heated tower the pure nickel is deposited in granular form.

Nickel is a brilliant metal approaching silver in color. It takes a good polish and does not tarnish or corrode in dry air at ordinary temperatures. The melting-point of nickel is about  $1500^{\circ}\text{C}$ ., its specific gravity is about 8.3, when cast, and 8.7, when rolled. Nickel, if attracted by a magnet, becomes magnetic but loses its magnetism when heated to  $340^{\circ}\text{C}$ . It is almost as hard as soft steel, far more malleable, and when rolled and annealed, is somewhat stronger and almost as ductile. Nickel is rendered brittle by the presence of small percentages of carbon, arsenic, nickel oxide and sulphur. Small amounts of magnesium render it more ductile and iron makes it hard. The tenacity of the metal ranges from 75,000 lb. per sq. in. for thin sheets of annealed nickel to twice that value for very fine hard-drawn wire.

Nickel is used chiefly in making nickel steel, coins, German silver, resistance wires, and in plating.

*Monel metal* is an alloy carrying about 66 to 68 per cent nickel, 2 to 4 per cent iron, 2 per cent manganese, and the remainder copper. It can be cast, forged, rolled, drawn into wire, electrically welded, soldered or brazed and is easily machined. The melting-point is  $1360^{\circ}\text{C}$ . and the specific gravity in cast form is about 8.87. The shrinkage of castings in cooling is about the same as for steel,  $\frac{1}{4}$  in. per foot. Monel metal has about one-fifth the heat conductivity of copper and one-twenty-fifth of its electrical conductivity. In appearance it is not distinguishable from nickel. The tensile strength of Monel metal in castings is 60,000 to 80,000 lb. per sq. in., in hot-rolled bars 80,000 to 100,000 lb. per sq. in. The yield point is about 50 per cent of the ultimate strength in cast metal, and 75 per cent in rolled form. In ductility it compares favorably with soft steel, the elongation in 2 in. ranging from 18 per cent in cast metal to 40 per cent in rolled rods. The modulus of elasticity is approximately 23,000,000 lb. per sq. in.

Monel metal is highly resistant to corrosion and the action of sea water. This valuable property coupled with its great strength, ductility and toughness make it a very useful metal for propellers, pump rods and pump linings, roofing metal and for castings and wrought parts which must withstand attack of sea water or mine waters. Inasmuch as the metal costs about ten times as much as the steel, extended use has been limited by the expense involved.

## BRASSES AND BRONZES

**792. The Brasses—Copper-zinc Alloys.**—The most valuable brass alloys contain from 60 to 90 per cent copper and 10 to 40 per cent zinc. The color of brasses ranges from a silvery-white for alloys carrying little

copper to a copper-red for those containing little zinc. The color is also affected by the rate of cooling.

Brass may be either cast or wrought. Brass for castings usually contains from 30 to 40 per cent of zinc. The addition of 2 or 3 per cent of tin serves to increase hardness, but it also lessens ductility. One or 2 per cent of lead renders brass more easily turned, filed and polished, but reduces the ductility and strength; while 1 to 6 per cent of aluminum materially raises the strength but may lessen the ductility.

In all cases, when melting copper, brass or bronze, great care must be exercised to keep the air from the metal, in order to prevent oxidation. This is done by covering the metal, in the crucible, with a thick layer of powdered charcoal. The copper is first melted alone, in a deoxidized flame, and then the scrap brass and zinc (previously melted, these fusing at a much lower temperature) are added and the whole stirred vigorously to effect a thorough mixing. Sometimes this mixing is done after the crucible is removed from the furnace. If it is done in the furnace, the dampers should be nearly closed to prevent an excessive heat, which would vaporize the zinc. If iron molds are used they should be heated and the interior surfaces coated with a mixture of resin (3 pts.) and lard-oil (1 pt.) to prevent adhesion. In pouring, the metal must be very carefully skimmed. The pattern should be made to allow a shrinkage of  $\frac{1}{4}$  in. per foot. For common castings green sand is used, but for fine work the molds are dried. The leaded brasses are poured quite hot and chilled rapidly, but the bronzes are poured at as low a temperature as possible and into heated molds in order to avoid contraction cracks.

The mechanical properties of cast brasses of all compositions are shown in Fig. 5. It will be observed that the commercial brasses are stronger and more ductile than either of their components, copper and zinc. Electrical conductivity for the brasses is, however, very much less than for copper, being only 0.20 for brass containing 70 per cent copper.

Brasses suitable for hot working, by forging, rolling or extruding, carry from 37 to 45 per cent zinc. Those which are to be extruded often contain 2 to 4 per cent of lead to make them flow easily through the dies. Lead, however, lessens the amount of reduction in working, which these alloys will withstand without cracking, makes the metal more porous and more susceptible to burning during melting. The brasses wrought into shape by cold working carry less than 40 per cent zinc, usually the ratio of copper to zinc runs between 2 to 1 and 3 to 1.

Brasses containing equal parts of copper and zinc are used principally for brazing brass goods. They have a very high crushing strength but are too brittle to be mechanically worked.

Alloys carrying 57 to 63 per cent copper, often called *Muntz Metals*, are used for bolts, rods, tubes and various extruded shapes. These alloys

may be hot-worked but harden considerably when cold-worked and are very liable to season cracking. When slowly cooled from a cherry-red temperature these alloys have a tenacity of 55,000 to 65,000 lb. per sq. in. with an elongation in 2 in. of 50 to 60 per cent. The limit of proportionality is low, being in the vicinity of one-third of the ultimate strength.

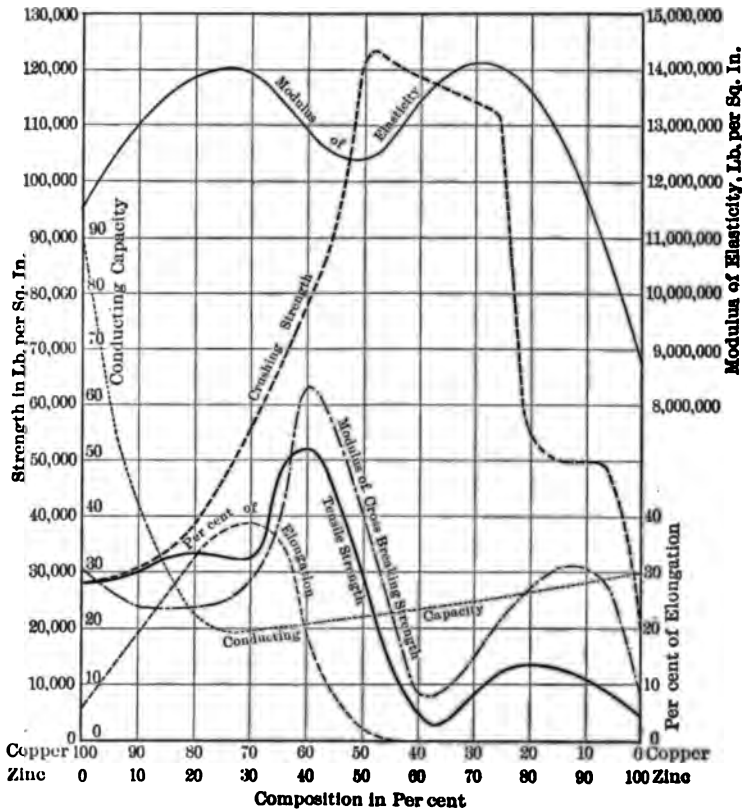


FIG. 5.—Properties of Cast Brass for Varying Proportions of Copper and Zinc. (Data from U. S. Test Board Rept., 1881, Vol. 2.)

The strength, hardness and ductility of these alloys appear to be somewhat increased by quenching.

Perhaps the most useful brasses are those containing approximately 2 parts copper to 1 part zinc, often termed "common" or "standard brasses." They are used for sheets, wire, and many stamped and drawn articles. A notion of the wide range of properties exhibited by these brasses may be obtained by comparing the properties of castings, Fig. 5, with the plate specimens of Fig. 6 and the hard-drawn specimens subjected to various annealing treatments, Fig. 7. Brasses of this composition annealed at



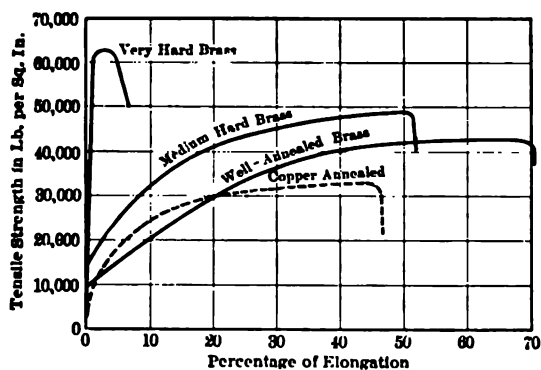


FIG. 6.—Stress Diagrams of Rolled Brass (Cu = 67 per cent, Zn = 33 per cent) and Copper. (*French Com. Rept.*, Vol. 3, Pl. 5.)

form, alloys of this composition are very ductile. Their strength and ductility are not materially changed by quenching. Cold working, however, raises the strength of the annealed brass from about 45,000 to 80,000 lb. per sq. in. but decreases the elongation in 2 in. from approximately 50 to 5 per cent.

With 80 to 95 per cent copper the brasses take a good polish and have a color somewhat like gold. They are used for medals, cheap jewelry and as gilding for percussion and primer caps. The tensile properties of a hard-drawn 90-10 brass under various annealing treatments are shown in Fig. 7.

**793. Complex Brasses** are alloys of copper and zinc with one or more other metals. The more important of these are manganese bronze, naval brass, sterro-metal, delta metal and Tobin bronze.

*Manganese bronze* is really a brass containing very small per-

a cherry-red heat have a Brinell hardness number of 45 to 55; when hard-drawn the hardness may reach 150 to 160.

Brasses carrying 70 to 75 per cent copper find use in cartridge cases, condenser tubes, tubes for brazing and in spinning operations. They have good resistance to corrosion and excellent cold working qualities. In cast

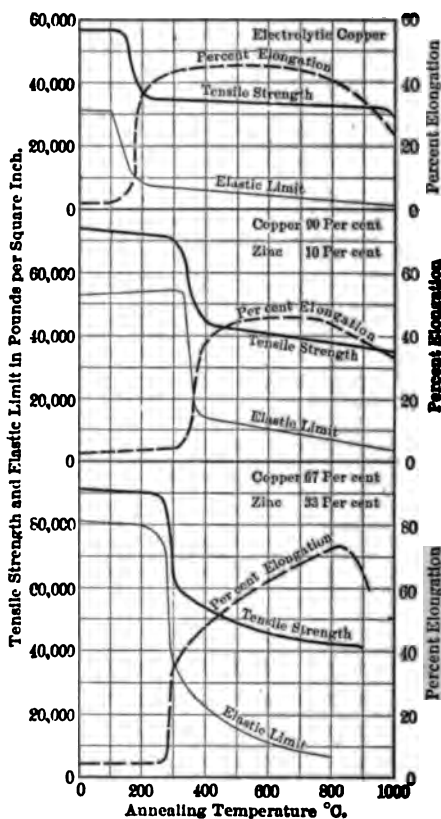


FIG. 7.—The Influence of Annealing on the Tensile Properties of Cold Rolled Copper and Brass (Grard, *Proc. I.A.T.M.*, 6th Congress II<sub>14</sub>.)

centages of tin, iron and manganese. The usual range in composition is about as follows: Copper, 57-62; zinc, 36-40; tin, 0.5-1.5; iron, 0.5-1.0; manganese less than 0.5 and lead under 0.2 per cent. Besides exercising a deoxidizing influence while in the bath of molten metal, manganese, by virtue of its association with iron strengthens, hardens, and slightly embrittles the brass. Tin also increases hardness.

The tensile strength of cast manganese bronze ranges from 70,000 to 80,000 lb. per sq. in. with an elastic limit of about one-third of the ultimate strength. The elongation in 2 in. generally lies between 20 and 30 per cent. In rolled or forged form its strength is slightly increased, its elastic limit raised to 30,000 or 40,000 lb. per sq. in. and the elongation in 2 in. is raised to 30 or 40 per cent. In compression the strength runs from about 90,000 lb. per sq. in. for castings to 150,000 lb. per sq. in. for rolled or forged parts. The modulus of elasticity of manganese bronze is about 16,000,000 lb. per sq. in.

On account of its high strength, the facility with which it may be forged or rolled, and its resistance to corrosion and salt water, manganese bronze is used considerably in marine engine parts, for hydraulic rams, cylinders, valve stems, propeller blades and bolts, and condenser tubes. Wrought manganese bronze parts are subject to season cracking and should always be annealed.

*Naval brass* is used for similar purposes and is much like manganese bronze in composition but lacks manganese and iron. It is slightly weaker and more ductile than manganese bronze. The tenacity in wrought form ranges from 55,000 to 70,000 lb. per sq. in. with an elongation in 2 in. of 30 to 45 per cent. Naval brass is also subject to season cracking and should be annealed when used in wrought form.

*Sterro metal* is an iron-brass containing about 38 per cent zinc with 1.5 to 2 per cent iron and the remainder copper. It has been used for hydraulic cylinders working under heavy pressures.

*Delta metal*, which is an improvement on sterro metal, is a proprietary composition, or brass, placed on the market since 1883 by a Mr. Alexander Dick (England). His process consists in incorporating a fixed amount of iron by making first a saturated solution of iron (about 5 per cent) in molten zinc. To prevent oxidation a little phosphorus is added to the melted copper. The proportions are varied for different purposes, having from 50 to 65 per cent copper, 50 to 30 per cent zinc, 0.1 to 5 per cent iron, and sometimes 0.1 to 1 per cent tin. This metal is as strong and ductile as mild steel, having a tensile strength, when rolled and annealed, of 60,000 to 80,000 lbs. per sq. in. with elongations in 8 in. of 40 to 14 per cent, respectively, at these limits.\* When cast in sand its tensile strength is 45,000 lb. per sq. in. with an elongation of 10 per cent. It is also highly resistant to corrosion.

*Tobin Bronze* is very similar to sterro metal and delta metal, the iron ingredient being somewhat less. Its composition is approximately 60 per cent copper, 38 per cent zinc, 1 to 2 per cent tin, with small portions (0.1 to 0.3 per cent) of iron and lead.

\* Tests made at Lloyd's Proving House, as given by Hiorns.

remarkable properties are due to rolling and annealing. As placed on the market, its tensile strength is from 60,000 to 80,000 lb. per sq. in., with an elastic limit of 60 per cent of its ultimate strength, and an elongation of from 25 to 15 per cent in 8 in. at these limits, respectively. It may be regarded as having the strength and ductility of structural steel, with the advantage of being non-corrosive. It can be procured in sheets from  $\frac{1}{8}$  in. to  $1\frac{1}{2}$  in. thick, and in round rods from  $\frac{1}{4}$  in. to 5 in. in diameter. Tobin bronze is readily forged at a cherry-red heat either by hand or by machinery, and also works well in the lathe. The elastic properties of this material are shown in Fig. 9.

**794. The Bronzes—Copper-tin Alloys.**—Since tin is added to copper principally to harden it,—it strengthens copper very little,—the copper-tin alloys may be regarded as a kind of hardened copper. The ancients used these alloys for their cutting-tools, and they are used now largely on account of their hardness and non-corrosive properties. The useful range of composition is 5 to 25 per cent of tin and 75 to 95 per cent copper.

In cast form the tensile strength of these alloys varies from 28,000 to 35,000 lb. per sq. in., maximum tenacity occurring for a tin content of about 18 per cent. The crushing strength of cast bronze rises from approximately 42,000 lb. per sq. in. for pure copper to a maximum of 150,000 lb. per sq. in. for bronze carrying 25 per cent of tin. The ductility of the bronzes is low. Cast bronzes carrying about 4 or 5 per cent of tin are the most ductile, elongating about 14 per cent in 5 in. With more than 5 per cent of tin the alloys lose most of their malleability when cold. The alloy having 12 per cent of the tin has an elongation in 5 in. of only 3 or 4 per cent and with 20 per cent tin it becomes practically nil. The tenacity also diminishes rapidly as the tin content of the alloy is raised above 20 per cent.

*Gun metal* is one of the strongest bronzes. It contains about 10 per cent tin and was formerly much used in casting guns. It is now used to some extent for strong castings. *Bell metal* is the hard sonorous bronze carrying about 20 per cent tin, used in making bells and gongs. By alloying 2 parts copper with 1 part tin a beautiful, hard, perfectly white metal is produced, called *speculum metal*. When polished this metal can be used for mirrors and reflectors.

**795. Complex bronzes,** are alloys of copper and tin with one or more additional metals. Among the more useful are zinc bronzes, phosphor bronzes, and lead bronzes.

*The Copper-tin-zinc Alloys.*—As shown in Fig. 8, the valuable ternary alloys are those in which copper forms the controlling element. This diagram is based on that principle in geometry which makes the sum of the normals from any point on the interior of an equilateral triangle equal to the altitude of the triangle. If the three altitudes be each taken as a scale of equal parts on which are indicated proportions (percentages) of copper, zinc, and tin respectively, these ranging from zero to 100, then to the same scale the sum of the three normals from any point in the triangle

will be 100, and hence these three normals may be used to indicate the percentages of the three metals which unite to form the alloy which is represented by that point in the triangle.\* An alloy of any two of these finds its place along one side of the triangle, of which the three apices make the 100-per-cent ends of the three metal scales. A little study of Fig. 8 will make this clear.

The contour-lines on this figure were drawn by J. B. Johnson after plotting on this triangle the tensile strengths of cast bronzes of known com-

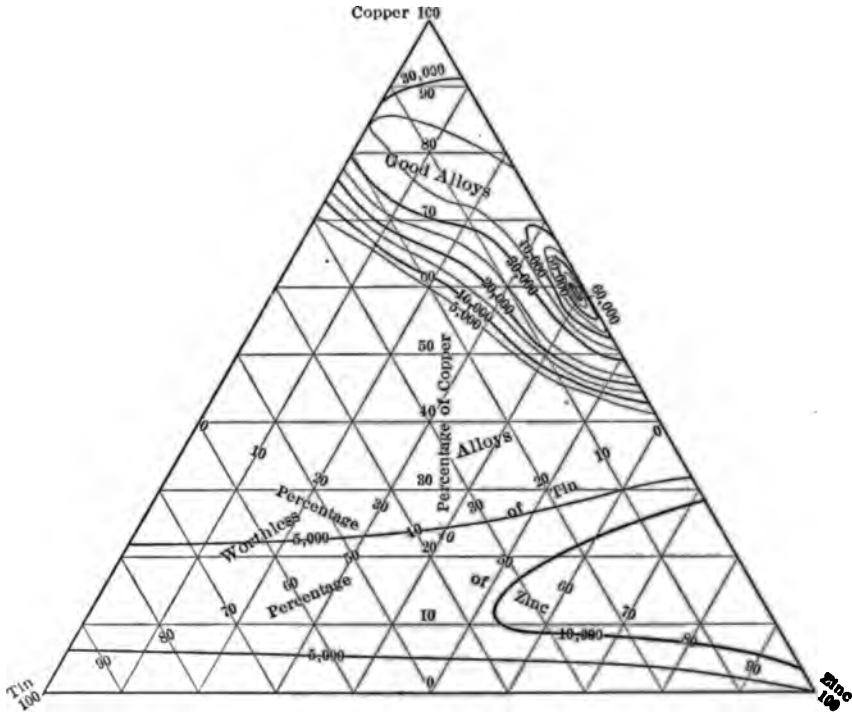


FIG. 8.—The Tensile Strength in Pounds per Square Inch of Copper-zinc-tin Alloys in Form of Castings. (Compiled by J. B. Johnson, from Records of U. S. Test Board, 1881.)

position from all reliable sources. From an examination of this chart it is at once evident that only those alloys near the copper apex are of any value, the strongest being, however, near the copper-zinc side, where the composition is about 59 per cent copper, 39 per cent zinc and 2 per cent tin. The tensile strength of such a casting, if properly made, is about 60,000 lb. per sq. in. It is too brittle, however, to be of much value. The most valuable alloys are those having an ultimate strength of from 35,000 to

\* This method of representing these triple alloys was first used by Dr. R. H. Thurston, *Trans. Am. Soc. C. E.*, 1881.

40,000 lb. per sq. in. tensile strength, with 20 to 30 per cent elongation. This is found in the vicinity of 75 to 85 per cent copper, 17 to 5 per cent zinc, and 8 to 10 per cent tin.

It must be understood, however, that so much depends on the purity of the ingredients and on the manipulation of the process of melting and casting, that this chart, or any similar record, must be taken as showing what *may* be obtained rather than what *will* be obtained from the use of these particular mixtures.

*Government bronze* or *Admiralty metal*, consisting of 88 per cent copper, 10 per cent tin and 2 per cent zinc, is a zinc bronze much used for valves, fittings, gears, and nuts where good strength is wanted. This metal has, when cast in sand molds, a strength of 30,000 to 35,000 lb. per sq. in. with a poorly defined yield point of 15,000 to 17,000 lb. per sq. in. The elongation in 2 in. is about 14 to 16 per cent. When annealed for a half hour at a temperature of 700 to 800° C. the ductility of the metal is much increased but the tenacity is practically unchanged. Government bronze cast in sand molds is considerably more ductile than that cast in chills. The ductility of the latter may be greatly improved, however, by annealing, at the above-mentioned temperatures.\* Government bronze is not a satisfactory metal for parts subjected to temperatures above 260° C.

*Phosphor bronze* is any copper-tin alloy made with phosphorus as a deoxidizer. Besides deoxidizing, it is also claimed that the phosphorus causes the tin to form a crystallized compound with the copper. It is mainly, however, as a cleanser of the melted metal from the oxide of copper that it is valuable. The phosphorus is added in the form of phosphor-copper or phosphor-tin, these containing phosphides of copper or of tin. For a malleable product, to be rolled or drawn into wire, the tin should not exceed 4 or 5 per cent, and the phosphorus should not exceed 0.1 per cent. For hard castings of great strength, as for pinions, valves, bearings, or bushings, use 7 to 9 per cent of tin and  $\frac{1}{2}$  to 1 per cent of phosphorus. A greater amount of phosphorus, up to 4 per cent, increases the hardness and brittleness. More than 4 per cent phosphorus will make the product useless.

Stress diagrams for phosphor bronze are shown in Figs. 9 and 10. The great toughness and high elastic limit of wrought phosphor bronze render it valuable for making springs which are subject to corrosion. The best grades of phosphor bronze carrying about 95 to 97 per cent copper,  $3\frac{1}{2}$  to 4 per cent tin and somewhat over 0.1 per cent phosphorus are used with good success in turbine blades.†

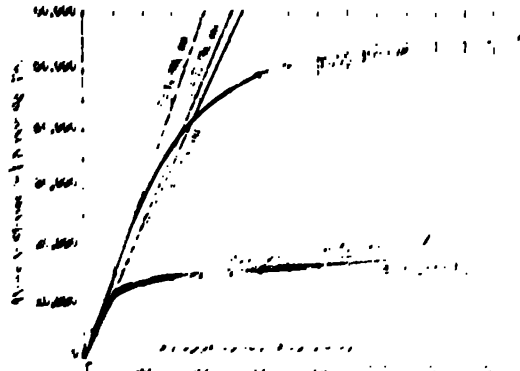
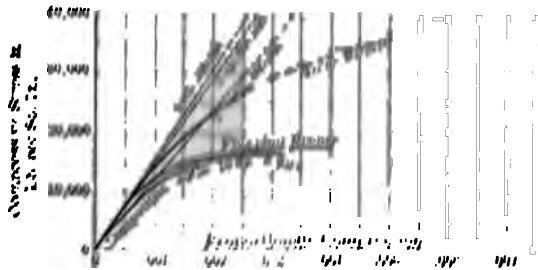
The United States Navy Department specifies for best grades of phosphor bronze castings a minimum tenacity of 45,000 lb. per sq. in. and an

\* See *Technologic Paper* No. 59, U. S. Bureau of Standards.

† *Jour. Inst. Metals*, Vol. 14, p. 56.

elongation in 2 in. of at least 20 per cent. The composition ranges for this metal are copper—85 to 90, tin = 6 to 11, zinc < 4, iron < 0.06, lead < 0.2 and phosphorus < 0.5 per cent. This metal is used for gears, driving and main shafts on steering gears and castings requiring free bearing qualities and resistance to corrosion.

Lead bronzes are alloys used for bearings. These have a composition of copper 80 to 90, tin 10 to 15, lead 1 to 5, zinc < 4, iron < 0.06, phosphorus < 0.5 per cent. These are used for bearings and castings requiring free bearing qualities and resistance to corrosion.



run hot. This metal is suited for turntable bearings and center bearings on swing bridges. For heavy trunnion bearings in contact with soft steel carrying less than the above pressure and operating at slow speeds the copper content may be raised to 83 or 85 per cent. A very common mixture for accurately lined machinery bearings is the 80-10-10 of copper, tin and lead with less than 1 per cent phosphorus. This bronze is easy to manipulate but heats and wears more than Dr. C. B. Dudley's "Alloy B," consisting of 77 per cent copper, 8 per cent tin and 15 per cent lead. On the other hand the lead in the latter alloy has a pronounced tendency to segregation during pouring and much skill is required to secure good bearings with it. It is used by the Pennsylvania Railroad for car journal bearings.

**796. Cold Working of Brasses and Bronzes.**—As with the ferrous metals so with the non-ferrous, cold working causes densification of the metal, decreases grain size, promotes an increase in tenacity, and decreases ductility. Thus Davis \* has shown that sheet brass (2 Cu to 1 Zn) varying from .01 to .06 in. in thickness increases in strength directly with the per cent reduction in area produced by rolling. For a reduction in area of 60 per cent the average strength was doubled, the per cent elongation in 2 in. was decreased to between  $\frac{1}{10}$  and  $\frac{1}{15}$  of the normal value, and the ductility as measured by the cupping test (Art. 798) was reduced to about  $\frac{1}{2}$  of its normal value. Further examples of the effect of cold rolling and cold drawing of the non-ferrous metals appear in Figs. 2, 6, 7 and 14. It is probable that cold drawing causes more dangerous internal deformation within parts which are to withstand tensile stress than cold rolling or hammering, since cold drawing produces tensile stress in the surface layers and compressive stresses within, whereas stresses of opposite character are set up by cold rolling or by hammering. The magnitude of the internal stresses so produced is quite variable ranging from 3000 or 4000 lb. per sq. in. to 50,000 or 60,000 lb. per sq. in.† When above 5000 or 10,000 lb. per sq. in. such stresses seriously impair the strength of the metal. Steps should, therefore, be taken to remove internal stress by annealing or springing as mentioned in the next article.

Although there are, in general, internal stresses—necessarily of opposite sign—induced by cold working, the results gotten by E. H. Peirce ‡ working on hard-drawn copper wire show that the hardening effect of such treatment is practically uniform throughout the mass of the metal and is not confined to the skin as has sometimes been stated.

**797. Season cracking** § is a type of failure common to rods, tubes,

\* *Proc. A. S. T. M.*, Vol. 17, p. 165.

† See *Technologic Paper No. 82*, U. S. Bureau of Standards.

‡ *Proc. A. S. T. M.*, Vol. 17, p. 115.

§ See E. Heyn in *Jour. Inst. Metals.*, Vol. 12, No. 2, p. 3; *Technologic Paper 82*, U. S. Bureau of Standards; *Topical Discussion in Proc. A. S. T. M.*, Vol. 18.

sheets, cartridge cases and other articles made of brass or bronze. It is especially prevalent in brasses containing 60 to 80 per cent copper, in manganese bronze, Tobin bronze, and has been observed in aluminum and aluminum bronze. It may take place when the part appears to be under no-stress; it may occur in a bolt or rod after it has carried a normal load for a considerable time, or it may happen when the brass is put into the annealing oven. The essential cause of "season cracking" is initial internal stress—produced by cold work, by burning in of defects, by quenching or by cooling strains induced in molding, coupled with one or more of the following:

- (a) Corrosion, the metal being under no external stress;
- (b) Applied stress of less intensity than the normal elastic limit;
- (c) Temperature changes.

Although acting in different ways these combinations effect an over-stressing at a certain region in the metal and cracking finally ensues. Considering the predominating cause of failure, "corrosion cracking" seems a more exact term to apply to the phenomenon.

Season cracking may be avoided by proper annealing; or by springing or by a combination of springing and annealing. Annealing at low temperature effects a removal of the greater part of the internal strain without producing material loss in hardness or strength. Indeed there is evidence to show that, in some cases, such annealing raises these properties.\* To be effective in preserving high strength and good ductility the annealing temperature must be less than the temperature at which grain growth begins in the metal. This temperature varies inversely (but not in direct ratio) with the amount of permanent deformation which the metal has suffered and with the time the metal is held at a given temperature. For brasses containing 2 parts copper to 1 part zinc the range of annealing temperature which is effective in relieving initial stress without diminishing hardness is 200 to 300° C.

Springing consists in bending a bar backwards and forwards in radial planes until all outer fibers have been permanently lengthened. This treatment reduces the intensity of tensile stresses which exist at the surface in cold-drawn bars, and is sometimes a necessary preliminary treatment to prevent cracking of parts in annealing.

**798. Special Tests for Brasses and Bronzes.**—On account of the large internal stresses present in many cold-rolled and hard-drawn objects of brass and bronze and the harm which may come from them, simple tests for their detection are needed. The most precise method with a rod or tube specimen consists in making accurate measurements of length on three or more surface elements, turning off or boring out a predetermined

\* Mathewson and Phillips, *Trans. A. I. M. E.*, Vol. 54, p. 608.



amount of metal and measuring again. Knowing the modulus of elasticity of the metal, the cross-sectional area before and after machining, and the original length, it is possible to compute the unit stress in each layer removed. Since this method necessitates taking a large number of readings with a comparator or strain gage and since the machining cuts must be very small and slowly and accurately done, it requires too much time for commercial purposes.

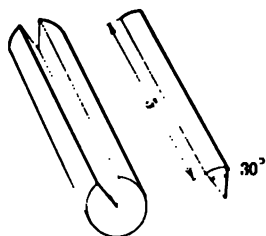


FIG. 11.—Miller's Specimen for Measuring Initial Stress in Brass Rods.

A more rapid method of measuring the internal stress in a bar has been devised by Mr. S. W. Miller. A specimen is cut from the straight bar, which is to be examined, as shown in Fig. 11. If the outer surface of the bar was in tension that surface of the specimen will be concave after its removal from the bar, the reverse would be true if the stress was compression. By measuring the mid-

ordinate to the curve, assuming circular curvature and the neutral plane passing through the gravity axis the change in unit stress on the outer and inner fibers may be computed from these formulas:

$$S_0 = \frac{8Efr}{3l^2} \quad \text{and} \quad S_1 = \frac{16Efr}{3l^2}.$$

Here  $S_0$  and  $S_1$  changes in unit stress at the outer and inner fibers respectively;

- $E$  = modulus of elasticity;
- $f$  = deflection at center of specimen;
- $r$  = radius of rod;
- $l$  = length of rod.

For thin sheets the procedure advocated by Merica and Woodward \* is still more simple. A strip about 6 in. long and  $\frac{1}{2}$  in. wide is coated with paraffin excepting one of the broad surfaces and etched with acid until one-quarter the depth is removed. If the strip curves the mid-ordinate of the arc is measured and the average value of the original internal stress is computed from

$$S = -\frac{36tEf}{7l^2},$$

where  $t$  = original thickness of strip,  $f$  = mid-ordinate of arc after etching, and other symbols have same meaning as before.

\* Topical Discussion on Season Cracking of Brass, *Proc. A. S. T. M.*, Vol. 18, Pt. 2.

Detection of the presence of high initial tensile stresses in the surface layers of a piece of brass may sometimes be accomplished by immersing the part in a solution of mercurous nitrate (65 gr.  $\text{HgNO}_3$  and 15 cc. conc.  $\text{HNO}_3$  per liter) for fifteen minutes to a half hour.\* Badly strained pieces will frequently crack after this procedure.

The *cupping test* has recently been strongly advocated † for determining the ductility and drawing qualities of thin sheet metal, especially brass. The test is made on a small sample of metal supported between a die and annular holder. A round-nosed tool is gradually forced against the specimen and forms it into a cup as shown in Fig. 12. By determining the depth of the cup at fracture, a direct measure of the drawing quality of the metal is obtained. Davis claims that this rapid test is far more accurate than the tensile test or scleroscope hardness test. It is also superior to the Brinell ball test for brass sheets under 0.05 to 0.08 in. thick. The Brinell test, however, is very satisfactory for determining the drawing qualities of thicker sheets. He also states that the roughness of the surface of the cup and the character of the fracture afford evidence of the grain size and will reveal defects.

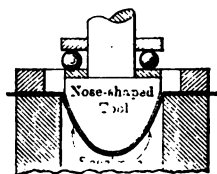


FIG. 12.—Showing Arrangement of Specimen, Tool and Dies on Erichsen Cupping Machine. (*Proc. A.S.T.M.*, Vol. 17, Pl. 2, p. 200.)

## ALLOYS OF ALUMINUM

**799. Utility of Aluminum Alloys.**—Owing to the softness of pure aluminum, it is commonly alloyed with copper or zinc to improve its mechanical properties. About one-fourth of the entire aluminum production is now utilized in making light, stiff alloys with these metals, a large portion of which are used in the automobile industry. Thus we find the light aluminum copper alloys used for pistons, cylinders and crank cases for both automobile and aeroplane engines, also for cooking utensils and strong light parts which are die cast. The light alloys of aluminum and zinc are used to less extent for gear cases, light castings of high strength, and for portions of scientific instruments requiring lightness and rigidity. With small additions of manganese and copper the aluminum zinc alloys have been used in the frames of Zeppelins. The heavy aluminum bronzes find use in steam valves, pump rods, spindles, springs, propellers, motor and engine gears where good strength and resistance to corrosion are essential qualities.

**800. Aluminum Bronze** is the rather inappropriate name applied to copper-aluminum alloys containing less than 11 per cent of aluminum. It

\* *Technologic Paper* 82.

† C. H. Davis, *Proc. A. S. T. M.*, Vol. 17, p. 165.

is made from the finest grades of copper and pure aluminum and is said to be improved by resmelting. On account of the rapidity with which aluminum oxidizes, the alloys must be smelted under a layer of charcoal and should be poured into the molds with as little agitation as possible. Castings must be provided with large gates and risers to provide against excessive shrinkage.

The ductility of the aluminum bronzes is very high and nearly constant for variations in aluminum content up to 7.3 per cent. Bronzes with more than 7.3 per cent aluminum show decrease in ductility as the aluminum increases and those containing 12 per cent are very brittle. The tenacity of these bronzes, however, increases directly with the aluminum content up to approximately 10 per cent aluminum. The tensile properties of these alloys \* are shown in Fig. 13. It will be observed that the ductility of small castings of these alloys is about the same as in the rolled rods. The bronzes containing less than 7.3 per cent aluminum are highly resistant to torsional stress and readily rolled, forged and cold-drawn. Since they consist of a solid solution, the rate of cooling influences their mechanical properties very little. Bronze with 7.3 per cent aluminum exhibited great toughness under impact and remarkable resistance to alternate bending stress.

Bronzes containing over 7.3 per cent aluminum contain, besides the solid solution, a dark needle-like constituent, which is unstable and much influenced by heat treatment. The marked influence of quenching on the properties of bronze with 10 per cent aluminum is also well shown in Fig. 13. This alloy in rolled form has about the same tenacity, elastic ratio, hardness, hardening capacity and toughness as 0.35 per cent carbon steel. Experiments by Corse and Comstock † have shown that cast rods of 10 per cent aluminum bronze quenched at 850° C. and annealed at 600° C. have an ultimate strength and elastic limit of approximately 95,000 and 45,000 lb. per sq. in., respectively, with an elongation of 10 per cent in 2 in. and a capacity to endure over 5,000,000 bending reversals producing an extreme fiber stress of 33,000 lb. per sq. in. Other tests by the Alloys Research Committee show that the 10 per cent aluminum bronze has almost constant strength for increasing temperatures up to 700° C. but the alloy is much more brittle above 400° C. than at normal temperatures.

The insertion of 1 per cent of manganese into a 10 per cent aluminum bronze increases the yield point and ductility of the alloy without producing material change in its strength or endurance under reversals of stress.‡

\* From Eighth Report of Alloys Research Com. *Proc. Inst. Mech. Engr.*, 1907. Pt. 1, p. 57.

† *Proc. A. S. T. M.*, Vol. 16, p. 134.

‡ See Alloys Research Com. Ninth Report *Proc. Inst. Mech. Engr.*, 1910, Pt. 1, p. 130.

Aluminum bronzes are almost incorrodible in sea water and are superior in this respect to Muntz metal or naval brass.

The modulus of elasticity for aluminum bronze is about 18,000,000 lb. per sq. in.

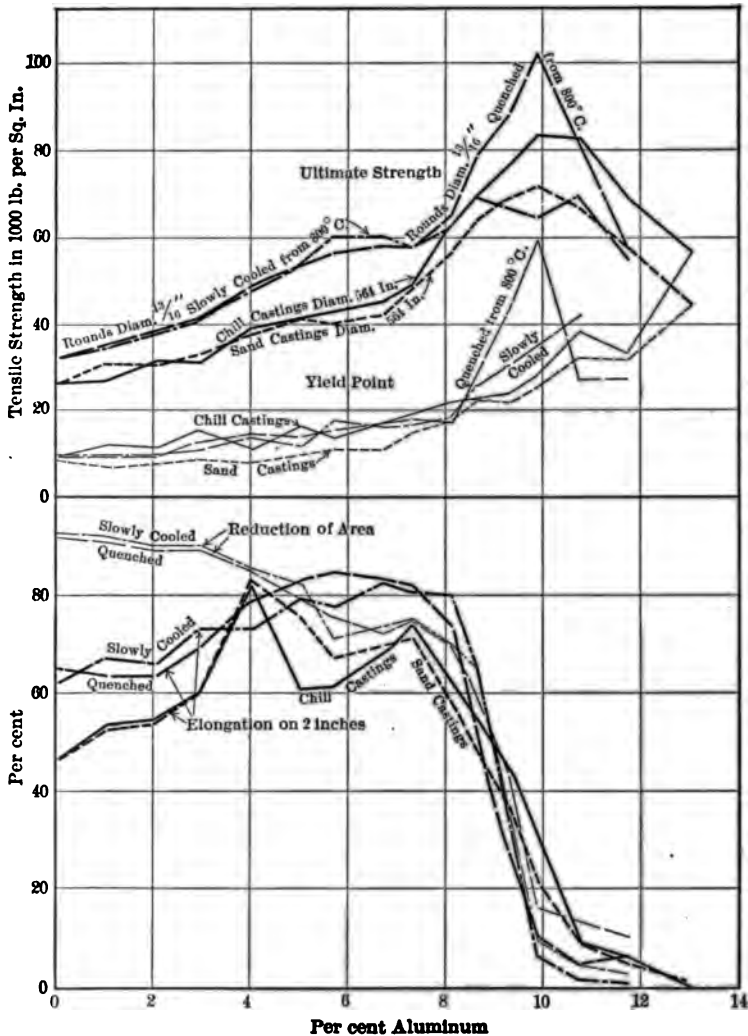


FIG. 13.—The Influence of Aluminum on the Tensile Properties of Aluminum Bronze. (Alloys Research Com., *Proc. Inst. Mech. Engr.*, 1907, pp. 113-133.)

**801. Aluminum-copper Alloys.**—By the addition of a small proportion of copper, generally less than 5 per cent, to “pure” aluminum, it is possible to secure a metal which is less liable to burning and to produce light castings that are stronger, harder and tougher than those made from aluminum.

Some of the mechanical properties of these alloys are listed in Table 3. They do not possess as high resistance to corrosion in sea water as the aluminum bronzes, nor are they as satisfactory as "pure" aluminum for cooking utensils. As mentioned before these alloys are much used in the automobile industry.

TABLE 3.—TENSILE PROPERTIES OF ALUMINUM COPPER ALLOYS

(From Alloys Research Com., 8th Report)

Per cent Copper.	Condition.	Diameter of Specimen. In.	Ultimate Strength. Lb. per sq. in.	Yield Point. Lb. per sq. in.	Per cent Elongation on 2 in. Per cent.
0.0	Sand cast. Slowly cooled.	0.564	10,250	4,256	19.0
1.90		.....	10,400	7,390	6.0
3.76		.....	16,840	10,960	4.0
4.97		.....	17,560	12,310	4.0
0.00	Sand cast. Quenched from 450° C. in water	0.564	11,300	5,375	22.0
1.90		.....	11,210	8,060	7.0
3.76		.....	15,790	12,760	4.0
4.97		.....	16,360	13,880	4.0
0.00	Hot rolled from 3 in. to 1½-in. rounds.	1.250	14,550	9,850	35.5
1.90		.....	24,400	14,320	27.5
3.76		.....	37,700	20,300	20.0
4.97		.....	32,850	19,920	19.5
0.00	Cold drawn.	1½	19,460	19,030	19.5
1.90		1½	33,600	31,370	13.0
3.76		1½	44,800	41,450	7.5
4.97		1½	39,700	35,600	6.0
0.00	1½-in. rounds. Annealed and drawn with annealing to 1½ in.	1½	16,600	15,660	23.0
1.90		1½	30,900	28,900	16.5
3.76		1½	37,850	34,700	8.0

**802. Aluminum-zinc alloys** containing less than 25 per cent of zinc are of the most commercial importance of the alloys of this group. As a class all of these alloys are very sensitive to high temperatures in melting and in solid form exhibit low strength and brittleness when heated above 50° C. Those having less than 15 per cent zinc are much used in constructions where a light, sound casting, which can be easily machined or forged into proper form, is desired. Alloys containing from 15 to 25 per cent of zinc are harder, stronger, but less ductile and more difficult to roll or draw. The alloys of high zinc content appear to suffer a decrease in strength when excessively worked either hot or cold. For example, the strength of alloys

containing 26 per cent of zinc decreased when the bars were hot-rolled or cold-drawn below  $1\frac{1}{4}$  in. in diameter.\* The tensile properties of these alloys are indicated in Fig. 14. Unlike most of the minor metals and their alloys the aluminum-zinc alloys have well-defined yield points characterized by dropping of the scale beam during test. The modulus of elasticity for all of the light alloys is approximately 9,000,000 lb. per sq. in.

Aluminum and zinc alloys containing about 5 per cent of aluminum are added in small percentages to the zinc baths used in galvanizing in order to render them more fluid.

**803. Aluminum - magnesium alloys (*magnalium*)** containing small percentages of magnesium, have been widely used as deoxidizers in copper smelting operations. The alloy containing 6 per cent magnesium has very good mechanical properties and is somewhat lighter than pure aluminum. It is easy to work, exceptionally strong, and ductile. Schirmeister † reports a tenacity of 42,000 lb. per sq. in., an elongation of 21 per cent and a Brinell hardness number of 69 for an alloy of this composition rolled at  $450^{\circ}\text{C}$ .

*Duralumin* is another light alloy of exceptionally high strength. It consists ‡ of aluminum alloyed with 0.5 per cent magnesium, 0.5 to 0.8 per cent manganese and 3.5 to 5.5. per cent copper. It has a tensile strength of 50,000 to 60,000 lb. per sq. in. and an elongation of about 8 per cent.

\* Tenth Report Alloys Research Com., *Proc. Inst. Mech. Engr.*, 1912, Pt. 1, p. 331.

† *Stahl u. Eisen*, June 24, 1915.

‡ *Mineral Industry*, 1915, p. 21.

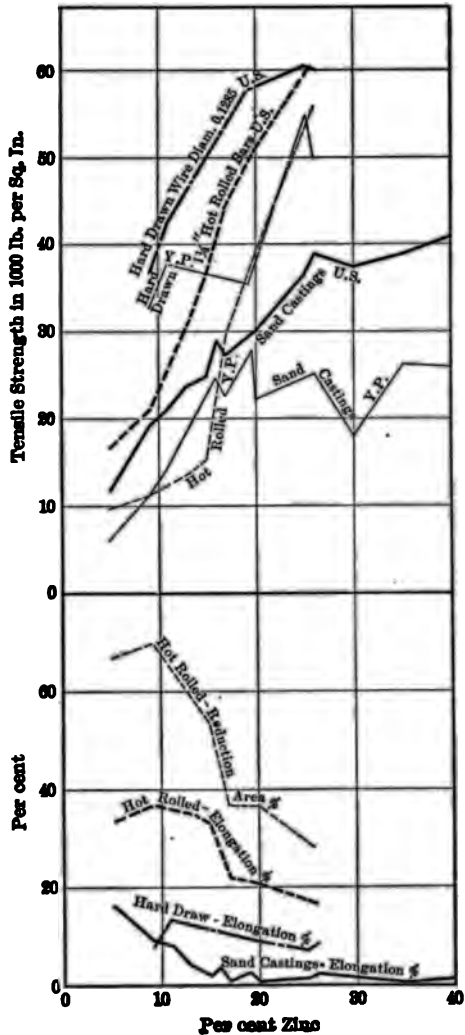


FIG. 14.—The Effect of Zinc on the Tensile Properties of Aluminum Zinc Alloys. (Alloys Research Com., *Proc. Inst. Mech. Engr.*, 1912, p. 449, 472, 494.)

## ALLOYS OF LEAD, TIN AND ANTIMONY

**804. Lead-tin alloys** are principally used in making solder, pewter and toys. These alloys have rather low melting-points, that of the eutectic containing 37 per cent lead being only  $183^{\circ}\text{C}$ . By adding tin to lead the strength and hardness are considerably increased. Furthermore, the alloys carrying more than 50 per cent lead remain pasty over a considerable range of temperature before completely solidifying. The latter property renders these alloys of value for plumbers' solder which ordinarily contains from 2 parts lead and 1 part tin to equal parts of each. For soldering tin, the alloy containing 2 parts tin and 1 part lead is much used, whereas for electrical work the best solders carry less than 10 per cent lead.

**805. Lead-antimony alloys.**—Antimony, like tin, serves as a hardener when added to lead but the useful binary alloys of these two metals cover only a limited range in constitution. The more useful of these alloys contain from 10 to 25 per cent of antimony. The alloys richer in lead than the above are very soft; those richer in antimony are too brittle and hard to be of value for most purposes. Alloys containing over 13 per cent antimony consist of a comparatively soft eutectic in which are embedded hard cubical crystals of antimony. In bearing metals the presence of a limited amount of these hard crystals increases resistance to wear, but a large proportion causes the bearing to heat. The softer eutectic, on the other hand, readily adjusts itself to inequalities in pressure and, wearing more rapidly than the antimony, allows space for the lubricant. Alloys containing 12 to 17 per cent antimony are inexpensive and are used to some extent for bearings carrying light loads. For properties of some bearing metals of this class, see Table 4.

Shot and bullets are made from lead containing a small proportion of antimony to increase the hardness.

**806. Lead-antimony-tin alloys** are of great value for type metal. A satisfactory type metal must take a full sharp impression of the mold and be sufficiently rigid to withstand press action. The alloys of lead and antimony containing over 15 per cent antimony satisfy the former requirement since they expand slightly on heating but they require a little tin to increase their compressive strength and render them less brittle. The range in composition for these alloys varies from 2 parts lead, 1 part antimony and 1 part tin to 7 parts lead, 2 parts antimony and 1 part tin. Linotype and monotype machines use cheaper alloys containing 12 to 18 per cent antimony, 3 to 8 per cent of tin, and the remainder lead.

Alloys containing 10 to 20 per cent antimony, 5 to 20 per cent tin and the remainder lead may also be used for bearings subjected to moderate loads (Table 4). Magnolia metal containing approximately 78 per cent

lead, 16 per cent antimony and 6 per cent tin is one of the common bearing metals of this class. The range in pouring temperatures for these alloys is, however, more limited than for the equally serviceable but somewhat more expensive bearing metals consisting principally of tin.

TABLE 4.—SHOWING PHYSICAL PROPERTIES OF WHITE METAL BEARING ALLOYS

(From Rept. of Com. B-2, *Proc. A. S. T. M.*, Vol. 18, Pt. 1.)

Alloy No.	FORMULA.				DEFORMATION OF CYLINDER 1½ in. Diam. by 2½ in. High at 70° F., in.			BRINELL HARDNESS.		Melt- ing- Point.	Com- plete Liqua- tion Point.	Wt.	Proper Pour- ing Temp- era- ture, deg. Fahr.
	Cop- per, per cent.	Tin, per cent.	Anti- mony, per cent.	Lead, per cent.	At 1000 lb.	At 5000 lb.	At 10,000 lb.	At 70° F.	At 212° F.	Deg. Cent.	Deg. Cent.	Os. per cu. in.	
1	4.5	91.0	4.5	....	0.000	0.001	0.015	28.6	12.8	225	371	4.24	824
2	3.5	89.0	7.5	....	0.0000	0.0015	0.0120	28.3	12.7	238	362	4.27	808
3	8.33½	83.33½	8.33½	....	0.0010	0.0045	0.0070	34.4	15.7	239	422	4.31	916
4	3.0	75.0	12.0	10.0	0.0005	0.0025	0.0090	29.6	12.8	185	291	4.35	680
5	2.0	65.0	15.0	18.0	0.0010	0.0030	0.0090	29.6	11.8	185	280	4.48	661
6	1.5	20.0	15.0	63.5	0.0015	0.0050	0.0180	24.3	11.1	185	267	5.39	638
7	...	10.0	15.0	75.0	0.0010	0.0050	0.0230	24.1	11.7	240	260	5.62	625
8	...	5.0	15.0	80.0	0.0020	0.0090	0.0620	20.9	10.3	243	260	5.80	625
9	...	5.0	10.0	85.0	0.0040	0.0120	0.0840	19.5	8.6	243	255	5.92	616
10	...	2.0	15.0	83.0	0.0010	0.0100	0.1540	17.0	8.9	245	260	5.82	625
11	...	...	15.0	85.0	0.0010	0.0100	0.1190	17.0	9.9	247	260	5.94	625
12	...	...	10.0	90.0	0.0025	0.0170	0.2850	14.3	6.4	247	265	6.17	634

807. Babbitt metals are alloys with a tin base containing small proportions of copper and antimony. They are so called from the inventor Isaac Babbitt. The important babbitts consist of crystals of the rather hard antimony-tin compound (SnSb) surrounded by softer masses of tin. Soft varieties of babbitt carry about 4 or 5 per cent of copper and 6 to 8 per cent of antimony, whereas the harder alloys contain 6 to 10 per cent of copper and 8 or 10 per cent of antimony. An idea of the mechanical and physical properties of the more valuable of these alloys is furnished in Table 4 abstracted from Report of Com. B-2 of *A. S. T. M.*, 1918. Since the rate of heating of such bearings under service conditions is dependent upon the size and distribution of the crystals of the copper-antimony compound, the temperature of pouring and rate of cooling must be carefully controlled. Rapid chilling or very slow cooling causes heating of the bearing.\* By heating the core of the mold it is possible to secure an intermediate rate of cooling such that the crystals will be well-defined, uniformly dispersed throughout the mass and not over .01 in. in size. Such treatment leads to a more serviceable bearing.

\* Behrens and Baucke, *The Metallographist*, Vol. 3.



**808. Alloys of Low Fusibility.**—The following remarkable alloys, all of which fuse at very low temperatures, may be used as safety-plugs in automatic fire-spraying pipe-systems in mills and for similar purposes.

TABLE 5.—FUSIBLE ALLOYS

Name.	PERCENTAGE OF INGREDIENTS.				Fusing Temperature, Deg. Cent.
	Bismuth.	Lead.	Tin.	Cadmium.	
Newton's.....	50	31	19	0	95
Rose's.....	50	28	22	0	100
Darcet's.....	50	25	25	0	93
Wood's.....	50	24	14	12	66-71
Lipoints's.....	50	27	13	10	60

## CHAPTER XXVII

### THE EFFECT OF TEMPERATURE ON THE MECHANICAL PROPERTIES OF METALS

#### EFFECTS ON IRON AND STEEL

**809. Importance of Temperature Effects on Properties.**—At the present time the ferrous metals are the principal materials for boilers, grates, fire-boxes, staybolts, engine cylinders and pistons, ladles, shafting, rails, axles, refrigerating machinery and other machine parts, which are subjected to considerable variation in temperature or to long periods at higher or low temperatures. Often the breakage of such parts would result in loss of life or large financial loss. It therefore is imperative that the properties of these metals under such conditions be well understood. We shall next consider the more important effects of temperature on the mechanical properties of iron and steel.

**810. Effects on Strength.**—This subject has been very fully and carefully investigated at the Watertown Arsenal, and a full series of stress d'agrams, similar to that of Fig. 1, is shown in the report for 1888. The curves of Fig. 1 exhibit the action of a structural steel (0.20 per cent carbon) having a normal tensile strength at 70° F. of 70,000 lb. per square inch and an elastic limit of about 37,000 lb. per square inch. Elastic limits and ultimate strengths may be gotten from this figure. A better notion of the variation in strength of different carbon steels, wrought iron and cast iron with temperature, can be formed from Fig. 2. Study of Fig. 2 reveals that the ultimate strength of the steels and wrought iron does not vary greatly for temperatures between 0 and 300° F., there being a slight sag in strength at 200° F. As the temperature is raised above 300° F. the strength increases until a maximum 10 to 15 per cent higher than the normal strength is reached at 500° F. Further increase in temperature is accompanied by a proportionate reduction in tenacity until at 1600° F. the strength is only 15 or 20 per cent of the normal. Corroborative evidence is also furnished by the data in Figs. 4, 5, 6, and 10. For information regarding strength of alloy steels at high temperatures see Art. 815.

It is noteworthy that the tenacity of cast iron is not materially affected by variation in temperature up to 900° F. For increases in temperature above 900° F. the strength falls proportionately until at 1600° F. it is approximately 20 per cent of the normal value.

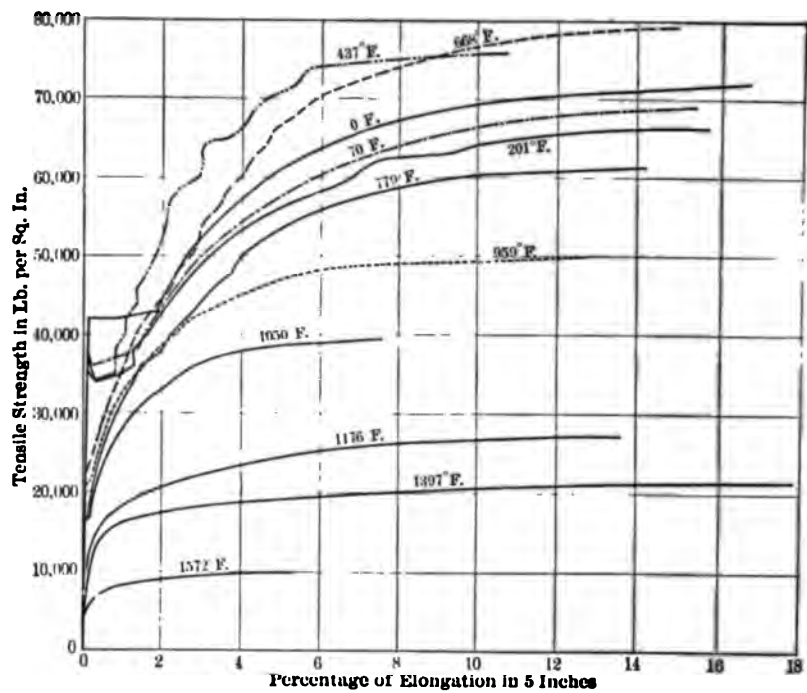


FIG. 1.—Stress-diagrams of Steel Bars at Various Temperatures. Carbon = 0.20%, Manganese = 0.45%. (*Tests of Metals*, 1888.)

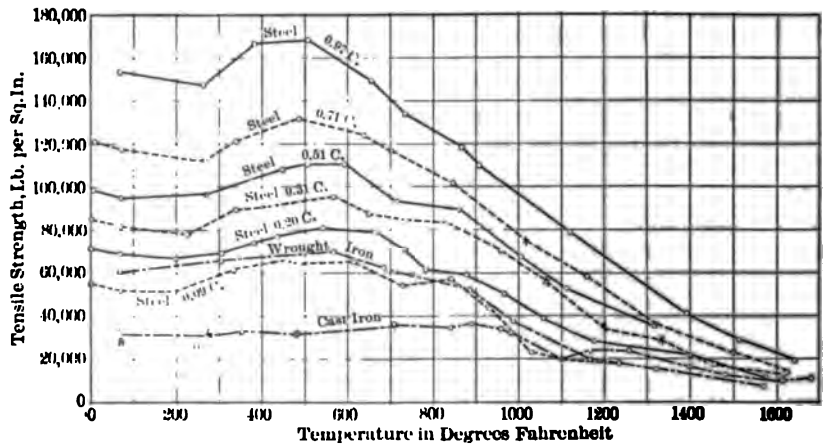


FIG. 2.—Variations in Tensile Strength of Ferrous Metals with Temperature. (*Tests of Metals*, 1888.)

In Fig. 3 may be seen the relative effects of slow and rapid applications of stress on wrought iron and steel at different temperatures. At ordinary temperatures the quick loading develops a greater tensile strength than the slow loading. Between 250° and 500° F. for steel, and between 150° and 500° F. for wrought iron (figure at left), the quick loading gives a lower ultimate strength, while beyond these higher temperatures the quick loading again gives the greater strength. Similar effects for both ultimate strength and yield point of soft steel wire, under temperature variations between -90° and +200 °F., are indicated in the right portion of Fig. 3.

When homogeneous steel or wrought-iron specimens are exposed to temperatures below the freezing-point of water the static strength is higher and the ductility lower than at normal temperatures. As an example we cite a test\* on a steel bar having a normal elastic limit and

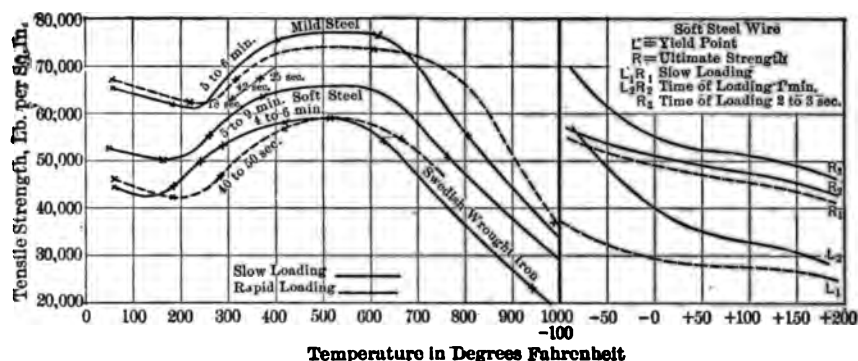


FIG. 3.—The Effects of Rate of Loading on the Strength of Steel and Wrought Iron when Tested at Various Temperatures. (*French Commission Report, Vol. 2, pl. 20.*)

ultimate strength of 52,800 and 72,400 lb. per square inch, respectively, and an elongation of 29.3 per cent. This bar tested at the temperature of liquid air (-220° F.) exhibited an increase of 51 per cent in elastic limit and 35 per cent in ultimate strength, but in elongation it showed a decrease of 63 per cent.

**811. Importance of Effect on Elastic Limit.**—In Fig. 4 combined curves for carbon steels showing mean variations in mechanical properties with temperature indicate that whereas the mean ultimate strength varies less than 15 per cent from the normal for temperature variations between 0 and 700° F., the mean elastic limit decreases in direct proportion to the increase in temperature above 0° F. Therefore the ratio of the elastic limit to the ultimate strength—the elastic ratio—is much less at high temperatures than at ordinary temperatures. For 100° F. the mean

\* Test made at Watertown Arsenal, See *Engr. Rec.*, Vol. 54, p. 65.

value of the elastic ratio is 0.57, while at 500° F. it is only 0.36. Figs. 1,

3, and 5, showing results of tests on individual classes of steel, substantiate the above statements.

For structural purposes, therefore, the working strength of wrought iron and steel must be regarded as regularly diminishing, while the temperature increases, the rate of diminution being about 4 per cent per 100° F. increase in temperature.

**812. The Change in Ductility.**—Steel and wrought iron both exhibit a marked reduction in per cent elongation for temperatures in the vicinity

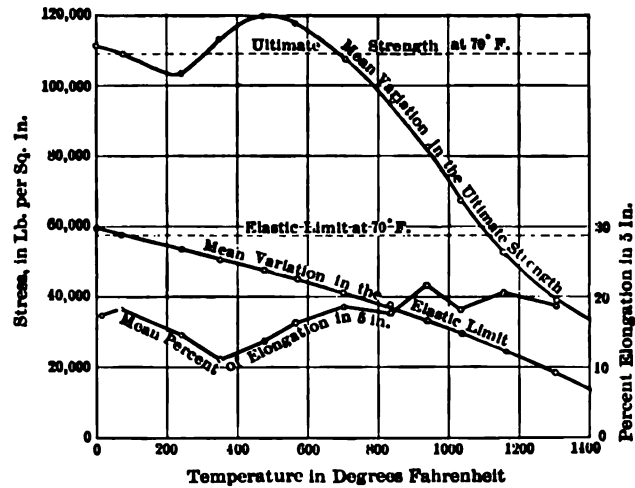


FIG. 4.—Grand Mean Curves from Temperature Tests on Steel Rods.

Diameter of rods, 1½ in., of specimen 0.8 in. Ten degrees of hardness, from 0.09 to 0.97% C. (*Tests of Metals*, 1888, p. 245.)

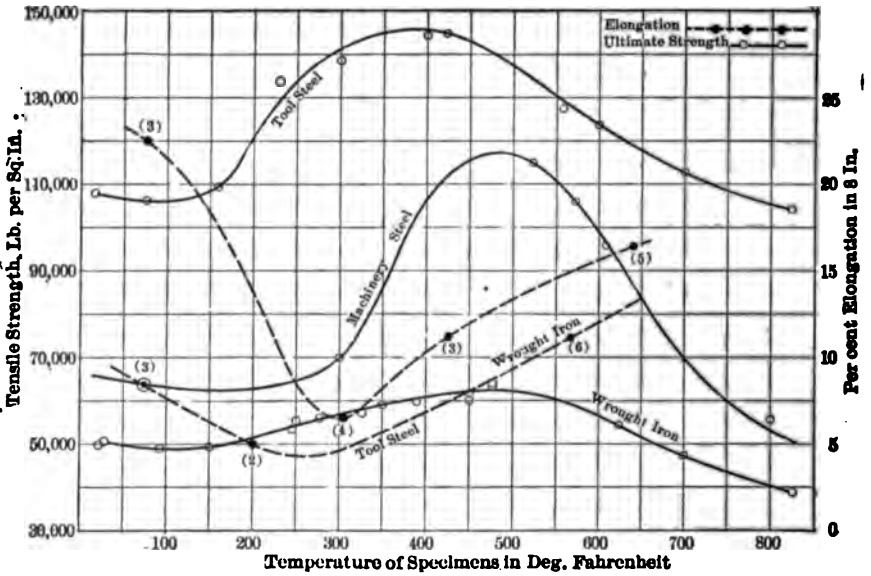


FIG. 5.—Variations in Strength and Ductility of Wrought Iron and Steel at Different Temperatures. (From Cornell University Tests in *Jour. West. Soc. Engr.*, Vol. 1.)

of 300° F., as the curves of Fig. 5 show. At this temperature wrought iron and soft steel have elongations of approximately one-third to one-half of the respective values at normal temperatures. The reduction in elongation at this temperature for high carbon steels is not quite so marked.

Minimum values for the per cent reduction in area of wrought iron and steel occur at somewhat higher temperatures, generally between 400 and 500° F., as indicated in Fig. 6.

**813. The Change in the Modulus of Elasticity.**—The results plotted in Fig. 6 and 7 show that the modulus of elasticity of wrought iron and of steel decreases as the temperature increases. This decrease amounts to approximately 2 per cent per 100° F. increase of temperature up to 500° F. For higher temperatures the reduction in stiffness is more marked, being about 20 per cent at 800° F. and 55 per cent at 1100° F. .

**814. Effect on Resistance to Impact.**—The variation in the resistance to a single blow of eight varieties steel tested at different temperatures is well shown in Fig. 8. The composition of these steels is given in Table 1.

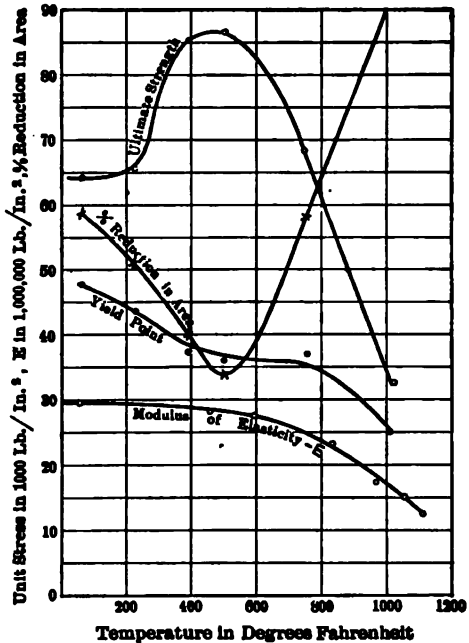


FIG. 6.—The Effects of Variations in Temperature on the Elasticity and Strength of Mild Steel.

(Lea and Crowther, *Engineering*, Vol. 98, p. 488.)

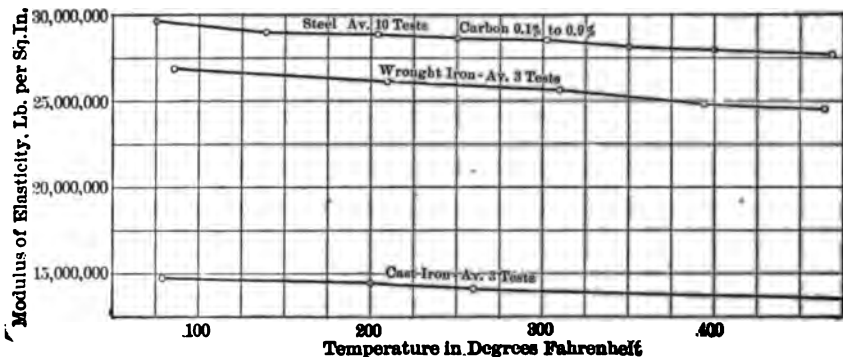


FIG. 7.—Effects of Moderate Variations in Temperature on the Modulus of Elasticity of Iron and Steel. (*Tests of Metals*, 1887.)

Each curve represents tests on 50 to 60 specimens. Each 0.4×0.4×2.4-in. test-piece was made with a milled notch 0.08 in. wide and 0.08 in. deep, the notch being provided with a round bottom. Specimens were broken in a Guillery \* machine over a 1.6-in. span and under an initial energy of

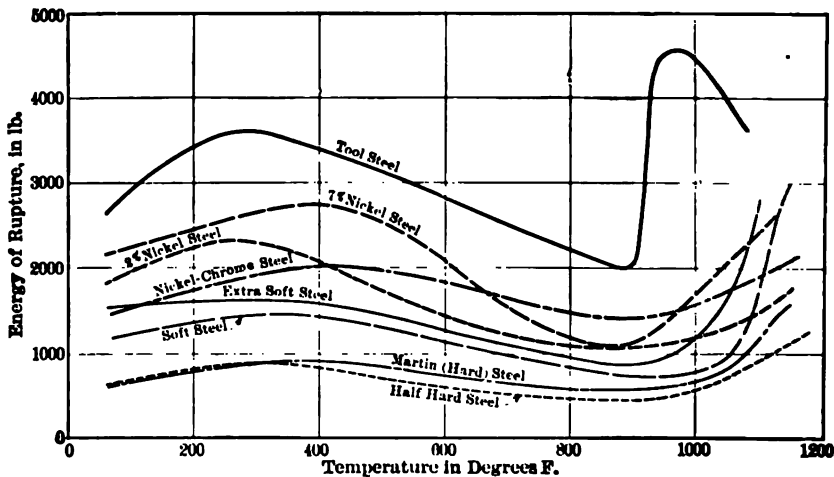


FIG. 8.—Effects of Temperature on the Resistance of Various Annealed Steels to Impact. (Guillet and Révillon in *Proc. I. A. T. M.*, 5th Congress, III<sub>4</sub>.)

TABLE I.—COMPOSITION OF STEELS SUBJECTED TO IMPACT AT VARIOUS TEMPERATURES. (GUILLET and RÉVILLON)

Steel	PER CENT OF					Annealing Treatment in Air at (°C.)
	C	Si	Mn	S	P	
Extra soft.....	0.218	0.26	0.24	0.041	0.013	850
Soft.....	0.345	0.20	0.51	0.048	0.068	800
Half hard.....	0.491	0.44	0.34	0.044	0.062	800
Martin (hard).....	0.725	0.40	0.33	0.025	0.013	750
Tool.....	1.224	0.05	0.04	0.040	0.023	{ Plane bars at 700°
2 per cent Ni.....	0.085	0.01	0.15	0.017	Ni=2.99	
7 per cent Ni.....	0.162	0.18	0.20	Ni=7.10	.....	600
Nickel-chrome.....	0.105	0.11	0.34	Ni=4.38	Cr=0.85	750

5210 in. lb. The tests show that the resistance of these steels increased slightly as the temperature rose from room temperature to a value between 250° and 400° F. The resistance to a single shock then diminished with increasing temperatures until a minimum resistance was reached at a temperature in the vicinity of 900° F., and that with further increase in temperature the resistance again rose.

\* This machine is provided with a fly wheel to which is attached a knife for breaking the specimens. The energy absorbed in breaking the test piece is measured by the diminution caused in the speed of the fly wheel.

Tests have demonstrated that repeated blows or working at temperatures between 500° and 600° F. (a blue heat) greatly impair the ductility and toughness of wrought iron and structural steel. Experiments by C. E. Stromeyer\* showed that the structural steel and wrought-iron bars, which in normal condition could be bent cold through an angle of 45° first on one side of axis of the test-piece and then on the other for 12 to 26 bends, stood only 1½ to 3 bends at a blue heat. Tension tests by R. Krohn † on wrought iron, soft steel, and medium steel showed that the decrease in energy of rupture due to working at a blue heat was 56, 30.4, and 26 per cent, respectively, while the elongation decreased 53, 31 and 28 per cent, respectively.

In certain experiments by T. Andrews ‡ wrought-iron car axles heated to various temperatures suffered impact from a 2240-lb. tup dropping through a distance of 30 in.; they were then turned over, the temperature restored and the blow repeated. Although the material itself was very non-uniform the results plotted in Fig. 9 demonstrate that the resistance was a minimum when the axles were tested at 570° F., a blue heat. Fig. 9 also shows the necessity of making impact tests at the same temperature if the results of different tests are to be compared.

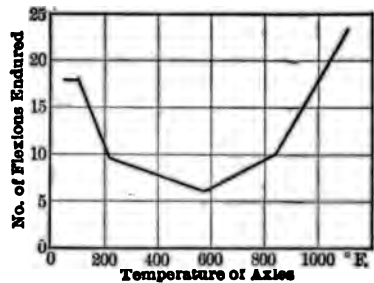


FIG. 9.—Effects of Temperature on the Endurance of Wrought Iron Railway Axles to Alternate Bending under Impact. (T. Andrews.)

Although comparatively few impact tests have been made at low temperatures, the available information indicates that the resistance to shock diminishes as the temperature drops below room temperature. Andrews found in other tests § on wrought-iron axles that the number of blows required to produce rupture at 100° F. was 50 per cent greater than at 0° F. Axles broken at -18° F. exhibited a crystalline fracture perpendicular to the axis of the piece; those broken at 300° F. showed a fibrous uneven fracture.

Impact tests by Goerens and Hartel || on mild steel show a great increase in brittleness over the results at normal temperatures when the specimens were broken at -103° F. and below.

The bar (Art. 810) tested under tension at the temperature of liquid air required approximately one-half as much energy to rupture it as at

\* *Proc. Inst. of Civ. Engr.*, Vol. 84, p. 114.

† Reported by G. Henning in *Engr. News*, Vol. 27, p. 42.

‡ In a Bessemer Premium Paper before the *Soc. Engr.* (London), 1896.

§ *Proc. Inst. Civ. Engr.*, Vol. 94, p. 209.

|| *Zeitschrift für anorganische Chemie*, Vol. 81, p. 130.



normal temperature, thus showing its increased fragility at low temperatures.

**815. Effect on Hardness.**—An elaborate series of tests by F. Robin (reported in *Rev. de Métal.*, Vol. 5, p. 893; Vol. 6, p. 180) in which the Brinell method was used, show that the curves between hardness and temperature are of the same general character as the curves between tensile strength and temperature.

TABLE 2.—BRINELL HARDNESS OF SPECIAL ALLOY STEELS AT VARIOUS TEMPERATURES. (ROBIN)

ALL STEELS TESTED AS RECEIVED EXCEPT AS NOTED

Kind of Steel.	HARDNESS AT				COMPOSITION IN %				
	-50 -58	100 212	400 752	800° C. 1112° F.	C.	Cr.	W.	Co.	Va.
High speed.....		655	515	480	.65	6.06	18.5	6.0	.....
High speed.....		540	465	400	.50	3.20	14.0	5.0	.....
High speed.....		415	350	315	1.00	8.00	20.8	Mo.	.....
Molybdenum (N).....	385	375	345	295	.30	.....	.....	2.0	.....
Molybdenum (H).....		640	430	290	.30	.....	.....	2.0	.....
Molybdenum.....	315	300	290	225	.30	.....	.....	0.5	.....
Vanadium (N).....	355	340	300	170	.80	.....	.....	.....	0.7
Vanadium (H).....		545	340	180	.80	.....	.....	.....	0.7
Tungsten (N).....		355	345	250*	1.2	.....	6.0	Ni.	.....
Tungsten (H).....		625	445	250	1.2	.....	6.0	.....	.....
Nickel.....		335	250	160	.....	.....	.....	16.0	.....
Nickel.....		165	135	80†	.....	.....	.....	27.0	.....
Chrome (N).....		350	320	230	1.18	1.37	.....	.....	.....
Chrome (H).....		600	415	160	1.18	1.37	.....	.....	.....
	50° C.	250° C.	400° C.	600° C.	C.	Mn.	Si.	P.	S.
<sup>1</sup> Carbon (N).....	290	320	285	180	2.02	.28	.25	.013	.017
<sup>1</sup> Carbon (H).....	350	400	325	...	2.02	.28	.25	.013	.017
<sup>2</sup> Carbon (N).....	265	280	245	160	1.08	.30	.12	.012	.020
<sup>2</sup> Carbon (H).....	600	520	325	110	1.08	.30	.12	.012	.020
<sup>3</sup> Carbon (N).....	205	210	225	...	.60	.72	.11	.022	.018
<sup>3</sup> Carbon (H).....	630	560	355	120	.70	.53	.34	.077	.053
Wrought iron.....	105	155	125	35	.04	.14	.11	.018	.020

N—as received. H=hardened. \* Estimated. † At 900° C. <sup>1</sup> Hardened in water at 1150° C., austenitic. <sup>2</sup> Hardened at 820° C. <sup>3</sup> Hardened at 850° C., martensitic.

Table 2 summarizes the more important results of Robin's work. The high, uniform hardness exhibited by the high-speed steels and by the molybdenum and tungsten steels at temperatures up to 1100° F. is in marked contrast to the hardness of the hardened carbon steels which were materially softened at 700° F. The high hardness of the 27 per cent nickel steel at 900° C. (1652° F.) is also noteworthy. Since certain comparisons by Robin indicate that the hardness is proportional to the strength for temperatures up to 100° C. (752° F.), this series of experiments furnishes important information concerning the relative strengths of alloy and ordinary carbon steels at high temperatures.

**816. Effect of Specific Gravity.**—Tests by Langley \* show that specific gravity of steel increases from 7.76 at white heat to 7.83 as the temperature falls to a black heat.

#### EFFECT ON ALLOYS AND METALS MUCH USED IN MACHINE PARTS

**817. Description of Tests.**—Owing to the extensive use of brasses, bronzes and other alloys in valves, shafting, parts of pumps and engines often subjected to temperatures from 400 to 700° F., a knowledge of the effects of such temperatures upon the mechanical properties of these metals is also essential if safe and economical designs of such parts are to be made. One of the most extensive series of tests thus far made on the effects of high temperatures on the mechanical properties of alloys was reported by Bregowsky and Spring to the Sixth Congress of the I.A.T.M.† We shall now present the main results of these investigators. Their experiments included both tensile and torsion tests. Round bars with the gauged portion turned down to a constant diameter of 0.65 to 0.85 in., depending on the size of bar as received, were used. The gauged length of the tension specimens was 2 in.; of the torsion test-pieces, 8 in. All metal was obtained from commercial foundries and all specimens for a given series of tests were from the same heat. Temperatures were secured through resistance coils which were placed around the specimens, due care being taken to insulate the specimen and to reduce the heating effect due to magnetic induction to a minimum. Each point on the diagrams represents from 2 to 10 results. In the tension tests the yield point was gotten by dividers; in the torsion tests a troptometer was used to measure detrusions. The compositions of the various metals tested may be found in Table 3.

**818. Effect on Tensile Properties of Alloys.**—Fig. 10 shows the effects of variations in temperature from room temperature to 1000° F. on 16 commercial alloys. It should be noted that both the ultimate strength and yield point of the copper-tin bronzes and the aluminum bronzes remain fairly constant for temperatures between 70° and 400° F. For increasing temperature both of these stresses decrease. The brasses, manganese bronze, and Monel metal exhibit a considerable falling off in strength as the temperature is raised above 70° F. It should also be noted that with the exception of manganese bronze and Monel metal the ductility of the brasses and bronzes is very little at temperatures above 600° F. The high strength and ductility exhibited by the rolled Monel metal at all temperatures is remarkable.

\* *Am. Chem.*, 1876.

† A good bibliography on this subject may be found in the *Proc. of the Fifth Congress of the I.A.T.M.*, entitled "The Influence of Increased Temperatures on the Mechanical Qualities of Metals," by M. Rudeloff.

TABLE 7. CHEMICAL ANALYSES OF ALLOYS AND METALS USED IN TENSILE AND TORSION TESTS OF STRUCTURAL STEEL WORK AND SPRING

		COMPOSITION IN PER CENT					
Mat.	Description	C	Si	Mn	P	S	P
Ferrous Metals used in Tensile Tests							
1	Cast iron, gray	2.5	0.2	0.2	0.02	0.005	
2	Cast iron, white	2.5	0.2	0.2	0.02	0.005	0.035
3	Steel	0.25	0.02	0.05	0.005	0.005	
4	Steel	0.25	0.02	0.05	0.005	0.005	
5	Steel	0.25	0.02	0.05	0.005	0.005	
6	Steel	0.25	0.02	0.05	0.005	0.005	
7	Steel	0.25	0.02	0.05	0.005	0.005	
8	Steel	0.25	0.02	0.05	0.005	0.005	
9	Steel	0.25	0.02	0.05	0.005	0.005	
10	Steel	0.25	0.02	0.05	0.005	0.005	
11	Steel	0.25	0.02	0.05	0.005	0.005	
12	Steel	0.25	0.02	0.05	0.005	0.005	
13	Steel	0.25	0.02	0.05	0.005	0.005	
14	Steel	0.25	0.02	0.05	0.005	0.005	
15	Steel	0.25	0.02	0.05	0.005	0.005	
16	Steel	0.25	0.02	0.05	0.005	0.005	
17	Steel	0.25	0.02	0.05	0.005	0.005	
18	Steel	0.25	0.02	0.05	0.005	0.005	
19	Steel	0.25	0.02	0.05	0.005	0.005	
20	Steel	0.25	0.02	0.05	0.005	0.005	
21	Steel	0.25	0.02	0.05	0.005	0.005	
22	Steel	0.25	0.02	0.05	0.005	0.005	
23	Steel	0.25	0.02	0.05	0.005	0.005	
24	Steel	0.25	0.02	0.05	0.005	0.005	
25	Steel	0.25	0.02	0.05	0.005	0.005	
26	Steel	0.25	0.02	0.05	0.005	0.005	
27	Steel	0.25	0.02	0.05	0.005	0.005	
28	Steel	0.25	0.02	0.05	0.005	0.005	
29	Steel	0.25	0.02	0.05	0.005	0.005	
30	Steel	0.25	0.02	0.05	0.005	0.005	
31	Steel	0.25	0.02	0.05	0.005	0.005	
32	Steel	0.25	0.02	0.05	0.005	0.005	
33	Steel	0.25	0.02	0.05	0.005	0.005	
34	Steel	0.25	0.02	0.05	0.005	0.005	
35	Steel	0.25	0.02	0.05	0.005	0.005	
36	Steel	0.25	0.02	0.05	0.005	0.005	
37	Steel	0.25	0.02	0.05	0.005	0.005	
38	Steel	0.25	0.02	0.05	0.005	0.005	
39	Steel	0.25	0.02	0.05	0.005	0.005	
40	Steel	0.25	0.02	0.05	0.005	0.005	
41	Steel	0.25	0.02	0.05	0.005	0.005	
42	Steel	0.25	0.02	0.05	0.005	0.005	
43	Steel	0.25	0.02	0.05	0.005	0.005	
44	Steel	0.25	0.02	0.05	0.005	0.005	
45	Steel	0.25	0.02	0.05	0.005	0.005	
46	Steel	0.25	0.02	0.05	0.005	0.005	
47	Steel	0.25	0.02	0.05	0.005	0.005	
48	Steel	0.25	0.02	0.05	0.005	0.005	
49	Steel	0.25	0.02	0.05	0.005	0.005	
50	Steel	0.25	0.02	0.05	0.005	0.005	
51	Steel	0.25	0.02	0.05	0.005	0.005	
52	Steel	0.25	0.02	0.05	0.005	0.005	
53	Steel	0.25	0.02	0.05	0.005	0.005	
54	Steel	0.25	0.02	0.05	0.005	0.005	
55	Steel	0.25	0.02	0.05	0.005	0.005	
56	Steel	0.25	0.02	0.05	0.005	0.005	
57	Steel	0.25	0.02	0.05	0.005	0.005	
58	Steel	0.25	0.02	0.05	0.005	0.005	
59	Steel	0.25	0.02	0.05	0.005	0.005	
60	Steel	0.25	0.02	0.05	0.005	0.005	
61	Steel	0.25	0.02	0.05	0.005	0.005	
62	Steel	0.25	0.02	0.05	0.005	0.005	
63	Steel	0.25	0.02	0.05	0.005	0.005	
64	Steel	0.25	0.02	0.05	0.005	0.005	
65	Steel	0.25	0.02	0.05	0.005	0.005	
66	Steel	0.25	0.02	0.05	0.005	0.005	
67	Steel	0.25	0.02	0.05	0.005	0.005	
68	Steel	0.25	0.02	0.05	0.005	0.005	
69	Steel	0.25	0.02	0.05	0.005	0.005	
70	Steel	0.25	0.02	0.05	0.005	0.005	
71	Steel	0.25	0.02	0.05	0.005	0.005	
72	Steel	0.25	0.02	0.05	0.005	0.005	
73	Steel	0.25	0.02	0.05	0.005	0.005	
74	Steel	0.25	0.02	0.05	0.005	0.005	
75	Steel	0.25	0.02	0.05	0.005	0.005	
76	Steel	0.25	0.02	0.05	0.005	0.005	
77	Steel	0.25	0.02	0.05	0.005	0.005	
78	Steel	0.25	0.02	0.05	0.005	0.005	
79	Steel	0.25	0.02	0.05	0.005	0.005	
80	Steel	0.25	0.02	0.05	0.005	0.005	
81	Steel	0.25	0.02	0.05	0.005	0.005	
82	Steel	0.25	0.02	0.05	0.005	0.005	
83	Steel	0.25	0.02	0.05	0.005	0.005	
84	Steel	0.25	0.02	0.05	0.005	0.005	
85	Steel	0.25	0.02	0.05	0.005	0.005	
86	Steel	0.25	0.02	0.05	0.005	0.005	
87	Steel	0.25	0.02	0.05	0.005	0.005	
88	Steel	0.25	0.02	0.05	0.005	0.005	
89	Steel	0.25	0.02	0.05	0.005	0.005	
90	Steel	0.25	0.02	0.05	0.005	0.005	
91	Steel	0.25	0.02	0.05	0.005	0.005	
92	Steel	0.25	0.02	0.05	0.005	0.005	
93	Steel	0.25	0.02	0.05	0.005	0.005	
94	Steel	0.25	0.02	0.05	0.005	0.005	
95	Steel	0.25	0.02	0.05	0.005	0.005	
96	Steel	0.25	0.02	0.05	0.005	0.005	
97	Steel	0.25	0.02	0.05	0.005	0.005	
98	Steel	0.25	0.02	0.05	0.005	0.005	
99	Steel	0.25	0.02	0.05	0.005	0.005	
100	Steel	0.25	0.02	0.05	0.005	0.005	
Alloys used in Tensile Tests							
1	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
2	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
3	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
4	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
5	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
6	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
7	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
8	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
9	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
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11	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
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13	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
14	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
15	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
16	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
17	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
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21	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
22	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
23	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
24	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
25	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
26	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
27	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
28	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
29	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
30	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
31	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
32	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
33	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
34	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
35	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
36	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
37	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
38	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
39	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
40	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
41	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
42	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
43	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
44	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
45	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
46	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
47	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
48	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
49	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
50	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
51	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
52	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
53	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
54	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
55	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
56	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
57	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
58	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
59	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
60	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
61	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
62	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
63	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
64	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
65	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
66	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
67	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
68	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
69	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
70	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
71	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
72	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
73	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
74	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
75	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
76	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
77	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
78	Aluminum alloy	0.25	0.02	0.05	0.005	0.005	
79	Aluminum alloy	0.25					

The behavior of the cold-rolled shafting and the cast steel is in accordance with the data of Art. 810 and 812. The 30 per cent nickel steel shows similar changes in strength properties but exhibits much higher ductility at temperatures under 600° F. than either of the other steels.

**819. Effect on Torsional Properties.**— Data on the strength and angle of twist under torsion is given for 15 metals in Fig. 11. With the exception of the carbon steels, all of the metals tested show a progressive falling off in torsional strength as the temperature is increased above 70° F. The curves for the carbon steels are somewhat similar to the tensile strength temperature curves. In these tests also, the Monel metal shows both high shearing strength and ability to withstand severe overstrain.

**820. Effect on Modulus of Elasticity.**—It is to be deplored that the effects of temperature on the stiffness of these metals was not noted. From tests made at the Berlin Testing Laboratory in 1893 it appears that the

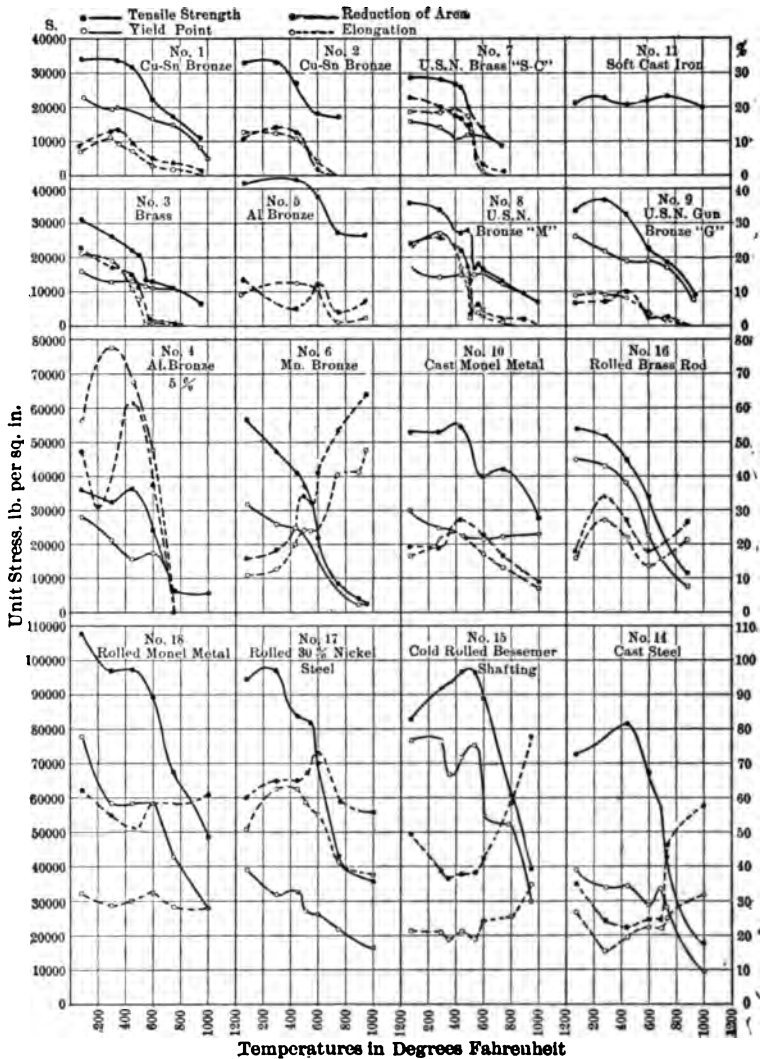


FIG. 10.—Effects of Variation in Temperature on the Tensile Properties of Different Commercial Alloys. (Bregowsky and Spring.)

modulus of elasticity copper and cast delta metal increase 10 to 15 per cent as the temperature increases from 70 to 200° F. and then fall off rapidly as the temperature is further raised, reaching room-temperature values again when the temperature reaches 400° F. Rolled delta metal

exhibited more pronounced diminution of stiffness with increase in temperature than did the cast delta metal. A 4 per cent manganese bronze had as high modulus at 600° F. as at 70° F.

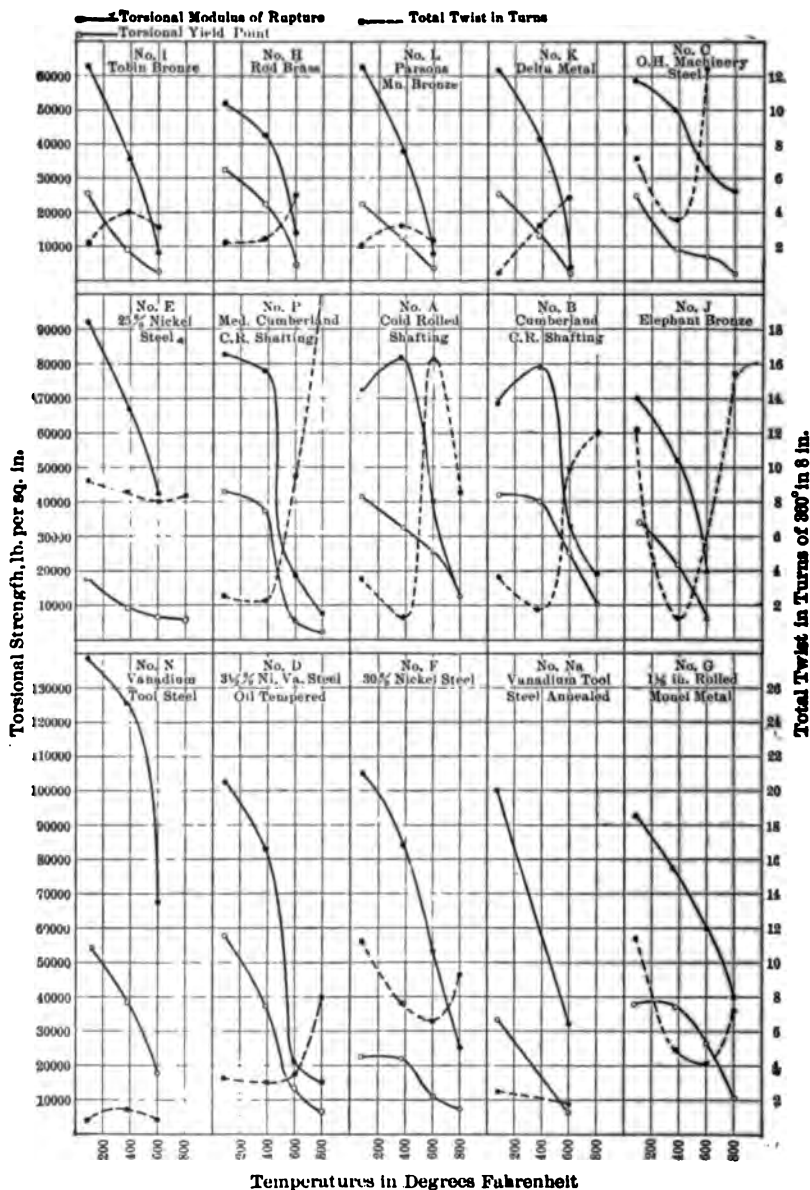


FIG. 11.—Effects of Variation in Temperature on the Torsional Strength and Angle of Twist of Commercial Rolled Alloys. (Bregowsky and Spring.)

## CHAPTER XXVIII

### FATIGUE OF METALS

By J. B. KOMMERS \*

**821. Fatigue Defined.**—Experiments have shown that metals will fail when loads are repeated or reversed many thousands or perhaps millions of times, not only at unit stresses less than the ultimate, but even at unit stresses less than the elastic limit. At one time it was supposed that these repetitions or reversals caused a general deterioration of cohesion in the metal, which deterioration was known by the term *fatigue*. It is now known, however, that no such general deterioration takes place, but that at the point of maximum stress certain “slip planes” are set up, some of which, upon continuous repetition of stress, gradually develop into actual cracks, which finally cause rupture. The word “fatigue” is not a very appropriate term for this class of failures. Alternating stress, or repeated stress failures are much better descriptive terms.

In the discussion of fatigue in this chapter, the term alternating stress or reverse stress will mean the case in which the stress changes from tension to compression. Repeated stress will be used as the broader term, including the previous case and also that in which the stress remains wholly tensile or compressive.

**822. “Crystallization” of Iron and Steel.**—It has often been observed that the fractures due to repeated stresses show a decided crystalline appearance, and this has given rise to the idea that the repeated stresses caused the crystallization. It is generally agreed now that this idea is incorrect. If such a bar, which has broken in service, shows a crystalline fracture, and another part of the bar is nicked and broken by a single blow, a crystalline fracture will again be revealed, proving that this condition is inherent in the bar, and not due to repetition of stress. If the structure of a member is coarsely crystalline due to defective composition, overheating, or other maltreatment, the member is likely to be weak under repeated or alternating stresses.

**823. Slip Lines and Fracture.**—As pointed out in Chapter XX, metals are composed of crystals, and in ordinary iron and steel the structure is by no means homogeneous. When a bar is deformed there will evidently be

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unequal distribution of stress among the crystals, and when the stress in the crystals which are most highly strained becomes sufficiently great they will give way by microscopic movement along their cleavage planes. This gliding is manifested by "slip lines" on the surfaces of the crystals.

In experiments on Swedish iron carried on by Ewing and Humphrey,\* the material had an ultimate strength of 52,800 lb. per square inch, and a yield point of 31,600 lb. per square inch. With the specimen acting as a rotating beam a unit stress of about  $\pm 20,000$  lb. per square inch produced slip lines on a few crystals after a few reversals. If the stress was comparatively high many crystals were affected. Under vertical illumination these slip lines appeared as fine dark lines. After more reversals additional slip lines appeared which had not been visible before, and the original ones showed a tendency to broaden. This broadening continued until some parts of the surface became almost covered with dark markings made up of groups of broadened lines. At this stage it was found that some of the crystals had cracked, the cracks occurring along broadened slip lines. In some cases the cracks were first seen on a single crystal, but they soon joined up from crystal to crystal until a long continuous crack was developed across the surface of the specimen, after which a few more reversals caused rupture. The effect of repeated sliding or grinding was seen at the polished surface of the specimen by the production of a burr, or rough and jagged irregular edge, broadening the slip band and suggesting the accumulation of debris. Even at unit stresses of only  $\pm 15,700$  lb. per square inch, slip lines appeared and then broadened, and the presumption is that these would ultimately have caused failure if the test had been continued.

A characteristic of the fracture of specimens which fail from fatigue is the complete absence of elongation and reduction of area at the break. The specimen fails abruptly, the fracture being at right angles to the direction of stress, in a manner ordinarily associated only with brittle materials.

**824. Experiments on Fatigue.**—In 1864 Sir W. Fairbairn published some results on a riveted wrought-iron girder subjected to repeated stresses. He included in his report some earlier experiments made by Captains James and Gaston. Fairbairn concluded from his experiments that it was not safe to use a repeated stress which was more than one-third of the ultimate strength.

In 1870 Wöhler published the results of a very exhaustive set of experiments which included repeated torsion, bending, and direct stress. The important conclusions drawn from Wöhler's experiments may be stated as follows:

1. Wrought iron and steel will rupture at a unit stress much below the

\* *Phil. Trans. Royal Society*, 1903, Vol. 200, p. 241.

ultimate strength and even below the elastic limit, if repeated a sufficient number of times.

2. Within certain limits, the range of unit stress, not the ultimate stress, determines the number of repetitions for rupture. (By range of unit stress is meant the algebraic difference between the maximum and minimum unit stresses applied.)

3. As the range is diminished, for a given maximum or minimum unit stress, the repetitions for rupture increase.

4. For a given maximum or minimum unit stress there appears to be a limiting range for which repetitions for rupture become infinite.

5. As the maximum unit stress increases the limiting range of stress diminishes.

These results will be better understood by a consideration of Figs. 1 and 2, and of Goodman's diagram shown in Fig. 3. Fig. 1 shows the rela-

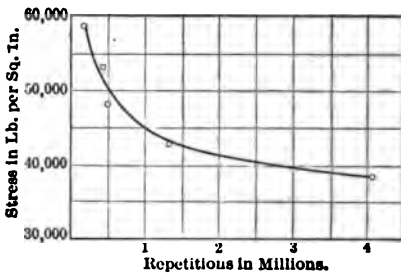


FIG. 1.

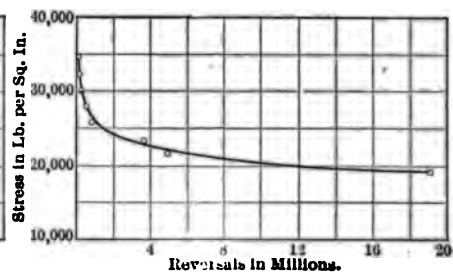


FIG. 2.

FIG. 1.—Wöhler's Fatigue Tests on Iron Axles, under repeated Bendings from Zero to the Stress Indicated.

FIG. 2.—Wöhler's Fatigue Tests on Rotating Bars, under Equal and Opposite Stresses. Range is Twice the Stress Indicated.

tion between unit stress and repetitions for failure when the maximum stress is as indicated and the minimum stress is zero. Fig. 2 shows the case of equal and opposite stress in a rotating bar. In engineering literature the term "limiting stress" is used to designate that unit stress at which the curve becomes practically horizontal. The limiting stresses in the figures would evidently be about 37,000 lb. per square inch in Fig. 1, and about 19,000 lb. per square inch in Fig. 2.

In Goodman's diagram, shown in Fig. 3, the minimum values of the limiting ranges are plotted as percentages of the static ultimate strength, forming the straight line *DOB*. The maximum unit stresses are then found to fall along a line *CAB*, which intersects the line *OE* at a point which is 50 per cent of the static ultimate. The values originally plotted by Goodman were for cases in which the material had withstood over 4,000,000 cycles of repeated stress.



It is evident from this diagram that as the maximum unit stress is increased the number of repetitions before rupture can be kept very large provided the range is decreased. If the maximum unit stress is 50 per cent of the ultimate, the repetitions before rupture can be greatly increased if the minimum stress is some value greater than zero. As the maximum unit stress approaches the ultimate strength of the material, the limiting range decreases. The figure also shows that for equal and opposite stresses

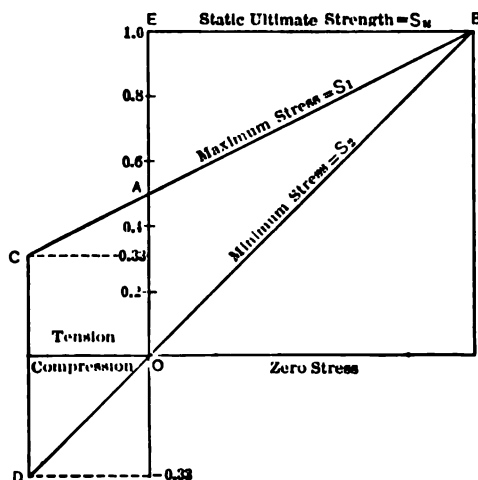


FIG. 3.—Goodman's Diagram.

in tension and compression the limiting stress is 33 per cent of the ultimate.

Since Wöhler performed his experiments many other investigators have carried on researches with the object of studying the phenomena further, and of determining the effect on the repetitions for rupture of such factors as heat treatment, speed of repetition, condition of the surface of the specimen, and variation in percentages of carbon and other ingredients. The relation of limiting stress to elastic limit and ultimate strength has

also been considered. A very complete bibliography of the work done up to 1913 may be found in the 1913 Report of the British Association for the Advancement of Science.

In Fig. 4 are shown characteristic curves obtained from results of fatigue tests. The curves are for a variety of materials and show what may be expected in repeated stress tests. The curves are plotted on logarithmic cross-section paper, and as has been pointed out by Basquin,\* the results plotted in this way fall on a straight line for values of repetition up to and slightly greater than 1,000,000.

**825. Effect of Heat Treatment.**—Results of experiments show that the fatigue strength of iron and steel is greatly influenced by heat treatment. Overheated steel is always weak, but can often be restored by reheating. Annealing in general decreases the endurance. Steel in the sorbitic condition seems to be especially strong against fatigue. It is certain, however, that the effect of heat treatment must always depend greatly upon the previous history of the material. From the few data that are available it would seem that when metal has been subjected to repeated stresses it

\* *Proc. Amer. Soc. Test. Mat.* 1910, Vol. 10, p. 625.

cannot be restored by annealing or by rest. These results are probably due to the fact that the metal had been so highly stressed or the number of

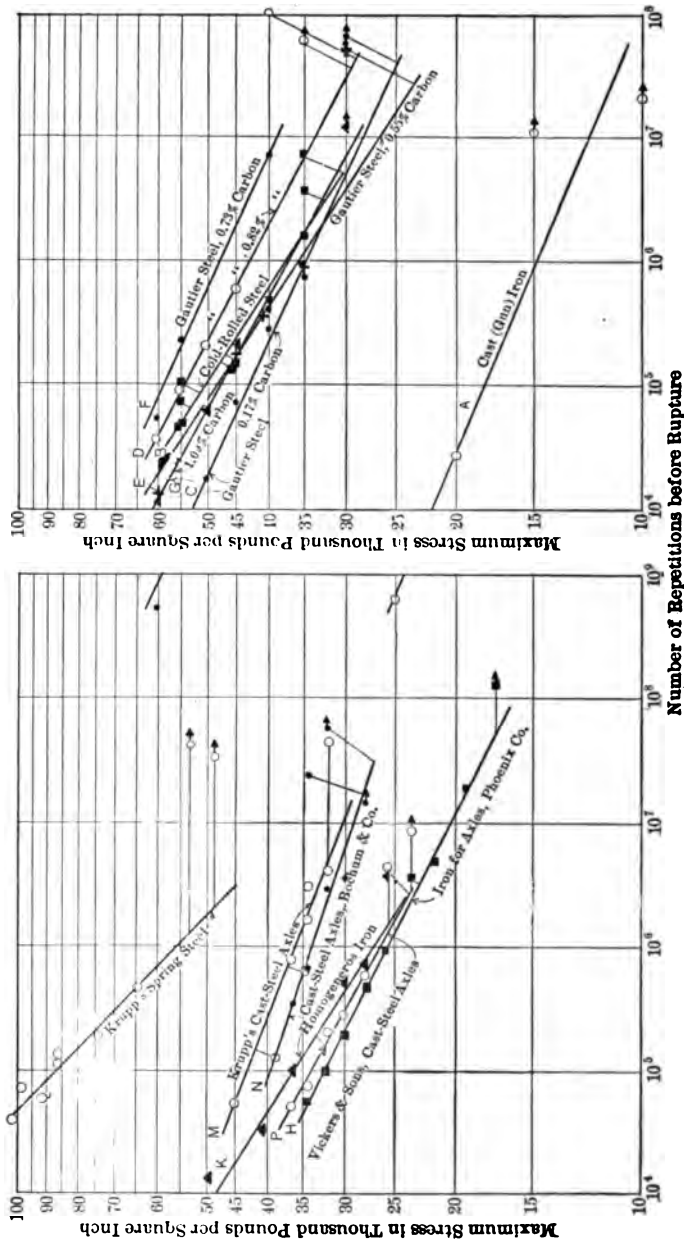


Fig. 4.—Stress-repetition Curves for Various Ferrous Metals Plotted to Logarithmic Scales.  
(Moore and Seely in *Proc. A. S. T. M.*, Vol. 15, p. 437.)

repetitions so large that slip bands or actual microscopic fissures had been formed which were not healed by annealing.

**826. Effect of Speed.**—The results of experiments are contradictory with regard to the effect of speed. Up to speeds of 2000 cycles per minute the effect of speed does not seem to be appreciable. At higher speeds it has been claimed that the endurance was increased, but the comparison was made from results on two quite different machines.

**827. Effect of Surface Condition and Change of Section.**—The endurance of metals is greatly influenced by the condition of the surface and also by the shape of the specimen. A polished or ground specimen will give higher results than one that is turned or which has a scratched surface.

Stanton and Bairstow \* have shown that a sudden change of section is a great source of weakness in withstanding repeated stresses. They obtained their best results on specimens which had fillets of 0.375 in. radius and diameters of 0.25 in. Screw-threaded specimens and specimens with fillets of 0.062 in. radius showed a strength of about 70 per cent as compared with the first specimens mentioned. Specimens with no fillets and square corners showed a strength of about 50 per cent.

**828. Effect of Composition.** Results show that in normal steels the strength against fatigue increases with the carbon content up to about 0.50 carbon. Heat treatment will, of course, greatly influence all results on carbon steels.

Ingredients such as nickel, chromium, and vanadium apparently increase the endurance, although this is a matter which has not been studied systematically.

**829. Relation to Elastic Limit and Ultimate.**—There has been considerable discussion as to whether the limiting stress is more dependent on elastic limit or yield point than on ultimate strength. A survey of the published results shows that while there is considerable variation, the results connecting limiting unit stress and ultimate strength are much more consistent than those connecting limiting unit stress and either elastic limit or yield point. A great many experiments have been carried on with equal stresses in tension and compression. These show in many instances that the limiting unit stress for this case is in the neighborhood of 33 per cent of the ultimate. For repeated tension from zero to a maximum value many results show a value of limiting unit stress approximately 50 per cent of the ultimate. These values correspond with those of Goodman's diagram.

**830. Tests beyond the Yield Point.**—In commercial practice the time and expense of conducting tests which require millions of applications of stress to rupture the specimen, are so great as to prohibit the use of such tests. For this reason a variety of machines has been devised in which the specimen is subjected to stresses beyond the yield point of the material.

\* See *Engineering*, 1907. Vol. 83, p. 505.

This test will not determine the limiting unit stress, and its main use is for the purpose of determining whether a specimen comes up to standard specifications for good material, and also to determine which of a number of steels would be the best to resist repeated stresses.

The question arises whether a material which gives good results in a test when stressed beyond the yield point will give equally good results for stresses below the elastic limit. This question has not been answered at the present time.

J. O. Arnold \* has shown that such an expeditious test will detect brittleness in cases in which the Wöhler test does not show any weakness of the material.

In some tests performed by Stead and Richards † on a 0.44 carbon steel, a direct comparison could be made between Wöhler's test and the test beyond the yield point. Calling the results for the normal steel unity in each case, the following comparison resulted:

Kinds of Steel.	Wöhler's Test.	Test beyond Yield Point.
Normal steel.....	1	1
Overheated steel.....	0.59	0.63
Reheated steel.....	1.45	1.47
Annealed steel.....	1.38	1.43
Sorbitic steel.....	2.46	2.54

In the Wöhler test the cycles for rupture varied from 850,000 to 3,500,000; while in the other test they varied from 2200 to 9300. In this particular case the comparisons by the two methods are so much alike that the tests suggest at least the desirability of more experimental work which will allow of similar comparisons.

**831. Rapid Methods for Determining Limiting Stress.**—J. H. Smith ‡ found that when a certain constant range of stress was applied to a specimen repeatedly, and the mean unit stress was gradually increased, that at a certain maximum unit stress the extensometer readings increased faster than the increase in mean stress. The range determined in this way was found to agree with the limiting range as determined by the more tedious Wöhler method.

The method which Smith finally adopted to determine this yield range was to apply a range of stress, first with zero mean unit stress. Keeping the range constant, a stress of a chosen amount was then added in tension and the extensometer reading was taken, then an equal amount of mean stress (range same as before) was added in compression and the extensometer reading was again taken. Following the above order of procedure the mean stress was thus gradually increased, the yield range being indi-

\* See *Engineering*, Vol. 85, p. 565.

† *Jour. Iron and Steel Inst.*, 1905, Vol. 68, p. 84.

‡ *Jour. Iron and Steel Inst.*, 1910, Vol. 82, p. 246.

cated when the extensometer readings were no longer the same in tension and compression upon reversal of the mean stress. By this method, starting with a small range of stress, a single specimen could be used to determine the yield ranges, and a complete diagram obtained similar to that of Goodman.

If this method of determining the limiting ranges is verified by further experiments it will prove extremely valuable in commercial practice.

C. E. Stromeyer \* has also suggested a method for an expeditious determination of the limiting stress. He allowed water to flow around the specimen, which was subjected to alternating torsion. By the use of thermometers which could be read to  $0.01^{\circ}\text{C}$ ., he measured the temperature of the incoming and outgoing water. By his method, the limiting stress is the maximum unit stress which will just generate heat in the test piece.

In his experiments to check this value, Stromeyer used alternations up to about 300,000, and it would appear that his method and his formula hold for values within 300,000 alternations. It is not shown that either his fatigue limit or his formula is correct for alternations say from 1,000,000 to 10,000,000.

If further experiments show this method to be correct for great numbers of alternations, it will be a very valuable method for commercial testing.

**832. Bauschinger's Theory of Failure.**—The only theory of failure due to repeated stress that has received serious attention is that propounded by Bauschinger.† He showed that the elastic limits in tension and compression as determined in the ordinary static tests were variable limits, which in many cases had been artificially raised or lowered by treatment during fabrication. He showed that when a bar was subjected to gradually increasing alternating stresses of equal amount in tension and compression, two equal elastic limits were set up in the bar—one in tension and one in compression. These he called the “natural” elastic limits, the elastic limits determined in the usual way being called the “primitive” elastic limits. If the tensile elastic limit was raised by alternating stress, the compressive limit was lowered a definite, but not necessarily the same amount. He showed that the elastic range found in this way under repeated stress was the same in magnitude as the maximum range of stress which could be applied repeatedly to a bar without causing rupture, even though the number of repetitions was very great.

Bauschinger found that the “primitive” elastic limits were unstable and only a few alternations of stress were necessary to produce the conditions in which the “natural” elastic limits obtained.

According to this theory, failure due to repeated stresses is caused

\* *Proc. Royal Society*, 1914, Vol. A90, p. 411.

† See Unwin's *Testing of Materials of Construction*, p. 361.

by the inelastic behavior of the specimen, so that a certain amount of work is done on the specimen during each cycle of applied stress. If stresses are kept within the "natural" elastic limits failure will not occur.

The results obtained by Bauschinger have been confirmed by Bairstow.\* In one of his experiments Bairstow used a specimen of axle steel which had a yield point of 55,700 lb. per square inch, and an ultimate strength of 85,500 lb. per square inch. When equal and opposite stresses of 31,600 lb. per square inch were applied, the cycle of extensions was represented for some time by the straight line, *A*, shown in Fig. 5. As the number of repetitions continued the curve changed into a hysteresis loop instead of a straight line, until at 18,750 alternations of stress the width of the loop was about 11 per cent of the elastic extension at the beginning of the test. The loop *B* in the figure, which represents the cycle of extensions at this time, was made up of straight lines, showing that no elastic limit in tension or compression was indicated by the extensometer. Evidently the natural elastic limits of the specimen had been exceeded, and Bairstow is of the opinion that at a slightly lower range of about 29,000 lb. per square inch, the specimen would have been perfectly elastic and no number of alternations would have produced a hysteresis loop.

When the stress was raised to  $\pm 33,600$  lb. per square inch, an immediate increase in the width of the hysteresis loop was produced. At  $\pm 45,200$  lb. per square inch and alternations of 23,280, a hysteresis loop like *C* in Fig. 5 was produced. The parts *FG* and *HE* were found to be parallel to the original straight elastic curve. *FG* was obtained while the tension was reduced to zero, and *HE* while the compression was reduced to zero.

The observations were continued almost to the breaking point, but the extensometer gave no warning of the deterioration of the specimen. Since the extensometer could detect extensions of 0.000002 in., the actual damage must have been extremely local.

Bairstow's results indicated that iron and steel, after a sufficient number of repetitions, were capable of adjusting themselves to variations of stress, cyclically applied. When this adjustment was complete, the

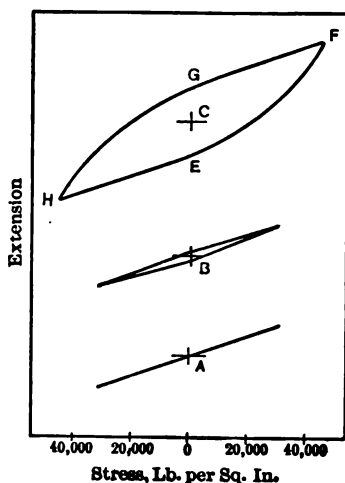


FIG. 5.—Cycles of Extensions.  
(Bairstow's Tests.)

\* *Phil. Trans. Royal Society*, 1910. Vol. A210, p. 35.



as fitting best the results obtained from numerous experiments. In this formula  $S_1$ =maximum unit stress,  $S'$ =repetition limit when  $S_2=0$ ,  $S_2$ = minimum unit stress, and  $S_u$ =ultimate static strength.

For the part of the field in which the stress is partly or wholly reversed Weyrauch suggested the formula:

$$S_1 = S' - \frac{S_2}{S_1} (S' - S'') \quad . . . . . (2)$$

in which  $S''$  is the unit stress which is wholly reversed and the other notation is the same as in the Launhardt formula.

**834. A Formula for Dimensioning Members Subjected to a Great Number of Repetitions.**—In Fig. 6 it would appear that a straight line for the upper limits of stress would fit the facts quite as well as the two formulas proposed by Launhardt and Weyrauch. There is no theoretical basis for these two formulas, and it would seem unreasonable to have a sudden change of law where the minimum stress passes through zero. By using a straight line, as shown in Fig. 7, a single formula is obtained which covers the field of both repeated and reversed stresses.

To derive the formula for the upper limit of stress the following values determined from experiment are made use of:

$$\begin{aligned} \text{Static stress limit} &= S_u = \text{ultimate strength;} \\ \text{Repetition limit} &= S' = \frac{1}{2} \text{ ultimate strength;} \\ \text{Reversal limit} &= S'' = \frac{1}{3} \text{ ultimate strength.} \end{aligned}$$

When the ultimate limits are reduced to working limits, suppose  $S'$  reduces to  $S_L$ , Fig. 7, and all other parts in proportion, giving:

$$\begin{aligned} \text{Working static load unit stress} &= 2S_L; \\ \text{Working live load unit stress} &= S_L; \\ \text{Working reversal unit stress} &= \frac{1}{3}S_L. \end{aligned}$$

To find the equation of the total working unit stress in terms of the maximum and minimum total unit stress on any member:

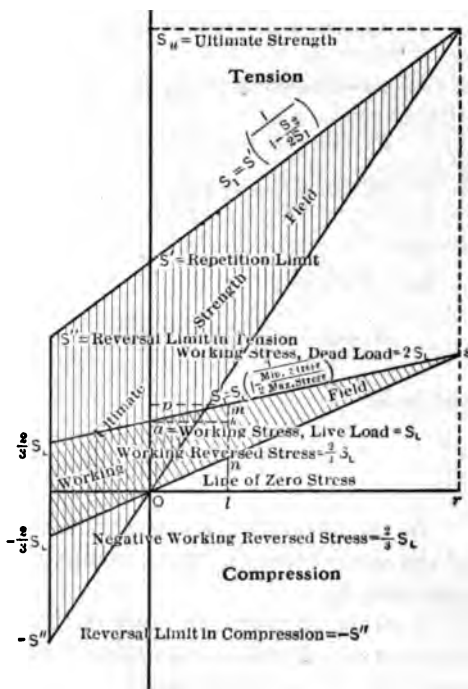


FIG. 7.



Let  $L$  = total live-load stress on any member;  
 $D$  = total dead-load stress on any member;  
 $A$  = area of cross-section of the member;  
 $S'_1$  = maximum stress in the member per square inch for both dead and live loads;  
 $S_L$  = working unit stress for live loads.

Then we have, from Fig. 7,

$$nl = \text{dead-load stress per square inch} = \frac{D}{A};$$

$$mn = \text{live-load stress per square inch} = \frac{L}{A};$$

$$ml = \text{total stress per square inch} = \frac{D+L}{A} = S'_1;$$

$$S'_1 = Op = Oa + hm \text{ and } hm = \frac{ln}{rs} (rs - Oa);$$

$$S'_1 = S_L + \frac{D}{A} = S_L + \frac{D}{2A}.$$

But  $A = \frac{D+L}{S'_1};$

$$S'_1 = S_L + \frac{DS'_1}{2(D+L)} = \frac{S_L}{1 - \frac{D}{2(D+L)}},$$

and finally

$$S'_1 = \frac{S_L}{1 - \frac{\text{Min. stress}}{2 \text{ Max. stress}}}. \quad \dots \dots \dots (3)$$

In this formula if the minimum unit stress becomes negative, the sign of the second term in the denominator changes to plus, thus making  $S'_1$  less than  $S_L$ .

Another argument in favor of the formula lies in the fact that it is the same as the old rule of using twice the factor of safety for live as for dead loads, as will now be shown.\*

\* In the case of bridge structures, the number of repetitions of the maximum stress is, in most cases, not high enough to involve the element of fatigue to any considerable extent, so that formula 3 is not properly applicable. Such structures are, however, subject to shocks and vibrations from the moving load which result in live-load stresses that are considerably in excess of the computed static values. It is, therefore, necessary in proportioning such structures to add to the computed static stress a considerable allowance for this dynamic effect, commonly called "impact," before applying the unit stress. The American Railway Engineering Association recommends that such impact allowance be determined by the formula  $I = \frac{100}{1 + \frac{L^2}{30000}}$ , in which  $I$  is percentage of static

live-load stress, and  $L$  is the length of the span in feet. It was formerly a common practice to use fatigue formula 3 for bridge structures, it being understood that such formula would take account of both impact and fatigue, and this practice is still followed to some extent; but the more recent practice is to make use of an impact formula and neglect the element of fatigue.

With the same notation as above:

$$A = \frac{L}{S_L} + \frac{D}{2S_L} = \frac{2L+D}{2S_L};$$

$$S'_1 = \frac{L+D}{A}.$$

Substituting the value of  $A$ :

$$S'_1 = \frac{2S_L(L+D)}{2L+D} = \frac{S_L}{1 - \frac{D}{2(L+D)}} = \frac{S_L}{1 - \frac{\text{Min. stress}}{2 \text{ max. stress}}}.$$

It is shown, therefore, that the best practice founded on experience, and the fatigue experiments, all agree and are all expressed in a formula which is universal in its application to stresses of the same and of opposite sign.

**835. Working Stresses by Diagram.**—The diagram shown in Fig. 8 was devised by John H. Barr \* of Cornell University. Its purpose is to evaluate formula (3) for both repeated and reversed loads, for various materials and various factors of safety.

- Let  $P$  = maximum load on member (of either sign);  
 $P'$  = minimum load on member (of either sign);  
 $S'_1$  = maximum intensity of stress for load  $P$ ;  
 $S'_2$  = minimum intensity of stress for load  $P'$ ;  
 $S_u$  = ultimate strength per square inch;  
 $S_w = \frac{S_u}{f}$  working stress per square inch for constant load;  
 $f$  = factor of safety.

Then the formula may be written:

For breaking loads—

$$S'_1 = \frac{\frac{1}{2}S_u}{1 - \frac{1}{2}\frac{S'_2}{S'_1}} = \frac{\frac{1}{2}S_u}{1 - \frac{1}{2}\frac{P'}{P}} \quad \dots \dots \dots (4)$$

For safe working loads for which  $S_w = \frac{1}{f}S_u$

$$S'_1 = \frac{\frac{1}{2}S_u}{1 - \frac{1}{2}\frac{S'_2}{S'_1}} = \frac{\frac{1}{2}S_w}{1 - \frac{1}{2}\frac{P'}{P}} \quad \dots \dots \dots (5)$$

In these formulas  $P$  and  $P'$  may be either of the same or of opposite signs. If of opposite signs, then the second term of the denominator becomes positive.

The use of the diagram will be illustrated by two examples.

*Example 1.*—Let the minimum load = 0.7 of the maximum load, and of the same sign; the ultimate strength of the material = 60,000 lb. per square inch; and let the factor of safety be 4. Find the maximum working stress.

Here  $\frac{P'}{P} = 0.7$ . Enter the diagram for this ratio, on the right hand side because  $P'$  and  $P$  are of the same sign (both tension, both compression, or both shear in the same direction), pass up on the 0.7 ordinate till the curve of factor of safety of 4 is intersected,

\* *The Sibley Journal*, Dec., 1901.

thence horizontally to the 60,000 lb. strength line, and thence upward to the scale of working unit stress at the top, determining a value of 11,500 lb. per square inch. This

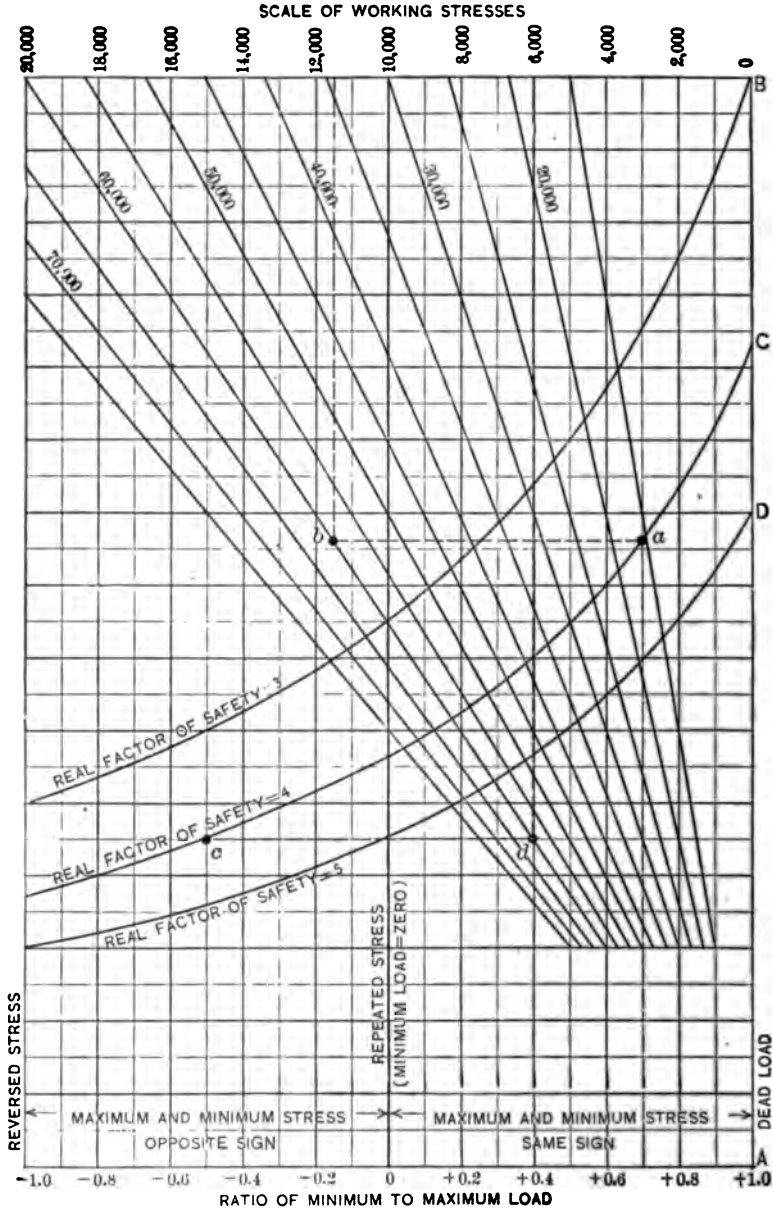


FIG. 8.—Diagram for Finding Working Stresses for Variable Loads. (Designed by John H. Barr.)

is the working unit stress for this material for a real factor of safety of 4, when 0.7 of the maximum load is removed and replaced an indefinite number of times.

**Example 2.**—For the same material, let 0.5 of the maximum load be imposed in the opposite direction an indefinite number of times, and find the working unit stress for a factor of safety of 4.

Enter the diagram for the ratio 0.5 on the left side, pass up to the factor of safety of 4, thence horizontally to the 60,000-lb. line, and thence vertically to the scale of working unit stress at the top, determining the value of 6000 lb. per square inch. This is the working unit stress for this material, for a real factor of safety of 4, when 0.5 of the maximum load is imposed an indefinite number of times in the opposite direction.

In using the Barr diagram it must be remembered that the diagram is based upon formula (3). Experimental results upon which this formula rests were determined from repetitions which were practically all within 10,000,000. The stress as determined from the Barr diagram with a factor of safety of 1 would be such a stress as would cause failure for repetitions within 10,000,000. Moore and Seely \* have found that a factor of safety of 1.8 corresponds to an increase in repetitions for failure of something over 100 times. A factor of safety of 2 on the stress as found from the Barr diagram should, therefore, ordinarily be ample to take care of the matter of repetition of stress. Judgment must determine the additional factor of safety necessary to take care of the possibility of sudden shocks or unexpected stresses.

As has been stated previously, Bairstow found that when the mean unit stress employed is tensile, there is a permanent elongation even when the maximum unit stress is less than the ordinary static yield point. For this reason, when using the Barr diagram for ratios of minimum to maximum stress from 0 to 1, no stress determined from the diagram should be used as a working unit stress unless it is well within the static elastic limit of the material.

**836. Formulas of Moore and Seely.**—Very often in practice in the design of piston rods, connecting rods, line shafts, turbine shafts, automobile axles, and many other machine parts, the number of repetitions of stress which a part may be called upon to withstand can be calculated. Moore and Seely † have developed formulas which may be used in such cases, one of the variables in the formula being the number of repetitions required for rupture.

The formulas are as follows:

$$S = \frac{B}{(1-Q)N^{\frac{1}{2}}}, \quad (6)$$

$$S = \frac{B}{(1-Q)N^{\frac{1}{4}}} (1 + 0.015 N^{\frac{1}{4}}), \quad . \quad . \quad . \quad . \quad . \quad (7)$$

\* *Proc. Amer. Soc. Test. Mat.* 1916, Vol. 16, p. 470.

† *Proc. Amer. Soc. Test. Mat.* 1916, Vol. 17, p. 470.

in which  $S$  is the upper limit of unit stress in pounds per square inch which will cause rupture after  $N$  repetitions of stress;  $Q$  is the ratio of the minimum stress to  $S$ ; and  $B$  is a constant depending upon the kind of material. Formula (6) is recommended for use in designing members whose failure would endanger life or limb; formula (7) for cases in which danger to life or limb is not involved.

The following tentative values of  $B$  have been suggested by Moore and Seely:

Material	$B$	Log $B$ .
Structural steel and soft machinery steel . . . . .	250,000	5.39794
Wrought iron . . . . .	250,000	5.39794
Steel, 0.45 per cent carbon . . . . .	350,000	5.54407
Cold-rolled steel shafting . . . . .	400,000	5.60206
Tempered spring steel . . . . .	400,000	5.60206
	to	
	800,000	5.90309
Hard-steel wire . . . . .	600,000	5.77815
Gray cast iron . . . . .	100,000	5.00000
Cast aluminum . . . . .	80,000	4.90309
Hard-drawn copper . . . . .	140,000	5.14613

In using these formulas the same precaution must be observed as was mentioned in connection with the Barr diagram; that is, no stress calculated from either formula should be used if it is greater than the ordinary static elastic limit.

To illustrate the use of the formula, suppose that a member is expected to withstand 10,000,000 repetitions of stress. Moore and Seely are of the opinion that a factor of safety of 100 as based upon the number of repetitions is not too large to use. Therefore, knowing  $B$  and  $Q$ , use 1,000,000,000 for  $N$ , and solve for  $S$ . An alternative method is to calculate  $S$  for 10,000,000 repetitions and then divide it by a factor of safety of about 2.

## CHAPTER XXIX

### THE CORROSION OF METALS\*

By O. P. WATTS†

**837. Importance to the Engineer of a Study of Corrosion.**—For nearly a century the problem of preventing the corrosion of metals has engaged the attention of scientific and practical men, and an enormous amount of experimental work has been done on this subject; yet we are still far from a satisfactory solution of the problem. With the enormous present use of metals this subject has become more important than ever before to every manufacturer and user of metals, to all owners of structures in which metal is largely employed, and to every engineer who has to deal with the construction or use of modern buildings or machinery. Even reinforcement deeply buried in concrete is not immune from corrosion. The statement of an eminent chemical engineer, that “rust taxes the people of the United States \$7,000,000 annually,” is probably an under- rather than an over-estimate. Over 60 per cent of the zinc produced in and imported into this country is used for galvanizing,‡ to fight rust. Each year sees a larger extent of metal surfaces exposed to corrosive influences, and therefore the tax levied on human industry by the corrosion of metals is continually increasing. When to the direct loss due to the destruction of metal there is added the cost of paints and other coatings used to prevent corrosion, the total tax chargeable to this source is stupendous.

Since the rusting of iron is the most important case of corrosion, and because it has been through the study of this phenomenon that our present conceptions of the nature of the process of corrosion of metals has been chiefly derived, it is fitting that this should receive special attention.

\* References: Sang's *Corrosion of Iron and Steel*. McGraw-Hill, 1910.

Cushman & Gardner's *Corrosion and Protection of Iron and Steel*. McGraw-Hill, 1910.

Friend's *Corrosion of Iron and Steel*. Longmans. 1911.

*Metal Corrosion and Protection*. A very complete bibliography. Published by Carnegie Library of Pittsburgh. 1909. 64 p. 10 cents.

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‡ W. A. Cook in *Metal Worker*, 1916, 85, 849.

**838. Great Variation in the Durability of Iron.\***—Cast-iron water pipe laid in France between 1664 and 1668 was in use in 1904, and probably still supplies water to the great fountains of Versailles. London and Glasgow have records of 120 years of service from cast-iron water pipe.

One of the most remarkable examples of the durability of iron is the column of Kuntab Minar erected at Delhi, India, 900 B.C., which is still in excellent condition after nearly 3000 years exposure to the weather. An American example of remarkable longevity in wrought iron was the suspension chains of the old bridge over the Merrimac River at Newburyport, Mass., which was removed in 1909 after 99 years of service. The chains were seemingly good for another century of use, in spite of the fact that they had been exposed to the weather without any protection for the greater part of this period.

On the other hand "in 1869 the British troop ship *Megaera* had to be beached at St. Paul's Island to prevent her sinking. Among other serious defects was a copper strainer fitted to a bilge suction pipe in a remote part of the ship. The action set up by it was sufficient to eat a hole right through the plates, and so admit water to such an extent that the ship had to be run ashore to save the crew." †

Cases of the corrosion of boilers and of the copper and brass tubes of the condensers of marine engines have been described frequently, but at too great length to be quoted here.

**839. Validity of the Acid Test for Determining the Relative Resistance of Metals to Corrosion.**—Until comparatively recently most experimental work on the resistance to rusting of various kinds of iron and steel was based on corrosion by acids; but it is now generally admitted that the acid test is not capable of determining the relative durability of different varieties of iron and steel under atmospheric conditions, and although this test will doubtless still be frequently employed, its findings will not be considered conclusive. The only reliable test appears to be that of subjecting the materials to actual working conditions, a process which usually defers the verdict for a number of years.

**840. Relative Resistance to Corrosion of Wrought Iron, Cast Iron, and Steel.**—In the last 75 years numerous tests have been carried out for the purpose of ascertaining the relative resistance to corrosion under various conditions, of wrought iron, cast iron, and steel. The most extensive of the early investigations are those of Mallet ‡ on the action of air and water on a hundred different makes of these three materials. Summaries of many later experiments may be found in Chapter 15 of Friend's *Corrosion of Iron and Steel*. From more recent experiments

\* *Mech. Eng.*, 34, 372; *Am. Soc. Civ. Eng.*, 1867 (1), 26; *Inst. Civ. Eng.*, 1853, 12, 487.

† Admiral Corner in *Jour. Inst. of Metals*, 1911, 5, 115.

‡ *Report of Brit. Assoc.*, 1839, 1840, 1843.

there appears to be little choice between good, modern wrought iron and steel as regards resistance to corrosion when exposed to the atmosphere, to sea water, or used as boiler tubes, or as pipes in water systems. Stahl \* finds that "steel and iron tanks for storing sulphuric acid last equally well."

The remarkable durability of cast-iron water mains has been shown by the examples already cited. In cases of corrosion by current straying from street railway tracks a similar leaching out of the iron often occurs, so that it is possible to drive a nail deeply into, or even entirely through a pipe that appears sound. Besides showing a lesser rate of corrosion both inside and outside, cast iron has an advantage over steel for water pipes in an initial thickness six or seven times as great. Another factor in the greater durability of cast-iron pipe appears to be the scale formed on the outside of the casting by contact with the sand of the mold. The removal of this increased two to four fold the rate of corrosion by air and by sea water.†

**841. Pitting.**‡—In the use of metal for roofing, pipes, tanks, etc., the way in which the corrosion is distributed over the surface may be quite as important a factor in determining its usefulness as the actual weight of metal removed or converted into rust. If corrosion takes the form known as pitting, it is evident that the removal of a very small amount of metal may end its usefulness; whereas, if the corrosion takes place uniformly over the whole surface, very many times this amount of metal may be removed before the apparatus fails to serve the purpose for which it was intended. In neutral or faintly alkaline solutions in which the rust initially formed remains attached to the metal, the rusting of wrought iron and steel almost invariably takes the form of pitting, i.e., deep corrosion confined to small spots here and there on the surface. An explanation of this peculiar type of corrosion is given under the effect of rust, Art. 849.

**842. Dissolved Air Stimulates Corrosion.**—It is generally recognized that the presence of air in solutions stimulates the rusting of iron and the corrosion of many other metals and alloys. In 1843 Mallet § said, "It would be desirable that the feed-water of marine boilers were heated to above 190° F. before entering them, and means provided for the escape of the air disengaged, which now enters the boilers and aids much in corrosion." Since that time the same recommendation has been reiterated again and again.|| Not only is the corrosion of iron by hot water greatly

\* Sang's *Corrosion of Iron and Steel*, p. 59.

† Thwaite, *Jour. Iron and Steel Inst.*, 1880, (2), 667.

‡ Matheson, *Jour. Iron and Steel Inst.*, 1909 (1), 105; Rosenhain, *Trans. Faraday Soc.*, 1916, 237.

§ *Rep. Brit. Assoc. for Adv. Sci.*, 1843, 12.

|| Walker, *Trans. Amer. Electrochem. Soc.*, 1908, 14, 186; Speller, *Mech. Eng.*, 1916, 37, 245.



lessened by removal of the air, but pipes carrying aerated cold water are often similarly benefited.

**843. Local Couples.**—Whenever two electrical conductors of unlike materials are in contact with each other in an electrolyte a voltaic cell is formed, and there is a flow of current in the solution between them, one being anode and dissolving, if it is soluble in that particular solution, and the other being the cathode by which the current leaves the electrolyte. The effect of such short-circuited voltaic cells, or "local couples," in accelerating the corrosion of ordinary metals and alloys has been recognized and studied for nearly a century.

An example of this is found in a report on the Panama Canal under date of January, 1915.\* "The top gate-valve seal is of cast steel and is held in place by bronze bolts. In practically every instance corrosion has been excessive around the heads of the bronze bolts, cutting away the metal and in some cases allowing the bolts to loosen and fall out."

**844. Purity a Factor in Corrosion.**—It is evident from a consideration of the corrosive effect on iron of contact with many other metals and alloys, that electro-negative materials present as impurities in iron and steel, or intentionally added as in making alloy steels, are likely to have an accelerative effect on corrosion. From this standpoint wrought iron of the highest grade would be expected to resist corrosion better than steel, and the superior endurance of ancient iron over modern steel has often been attributed to the greater purity of the former. Several manufacturers have adopted "purity" as their slogan, and have put on the market iron of exceptional freedom from impurities, claiming that their products resist rusting better than any other sheet iron or steel.

In direct opposition to the advocates of highest purity are those who add 0.1 to 0.25 per cent of the electro-negative metal, copper, to iron, claiming that experiment † shows such "impure iron" to resist atmospheric corrosion and attack by acids better than the pure irons. It is even contended that the remarkable durability of the iron of our forefathers, whose praises have been sung by many recent writers, owes its durability to the presence of a small amount of copper.‡

**845. Effect of Mill-scale on Rate of Rusting.**—One of the most commonly occurring negative materials which comes in contact with iron is the scale of black oxide which forms whenever iron is heated in contact with the air. This was early recognized as an accelerator of the rusting of iron, and its complete removal is specified, not only before painting or coating iron with some other metal, but whenever it is to be exposed to corrosive conditions. Although mill-scale itself is very resistant to

\* *Eng. News*, 1916, p. 1005.

† D. M. Buck, *Jour. Ind. and Eng. Chem.*, 1913, 5, 447; 1916, 8, 209.

‡ O. W. Storey, *Trans. Amer. Electrochem. Soc.*, 1917, 32, 211.

corrosion, it is brittle and therefore liable to be broken off in spots, and always contains cracks, so that when wet a short-circuited voltaic cell is formed, in which iron as anode corrodes more rapidly per unit of area exposed to the electrolyte than if the scale were not in contact with it.

Oxide dissolved in steel should also accelerate rusting. Speaking of specimens of steel polished for metallographic examination, some of which contain oxide and others do not, Law says:\* "If these are kept side by side in an ordinary room, it will be seen that the steels containing oxide begin to show signs of rusting long before the others, and in dilute acid solutions they corrode far more readily."

**846. Nature of the Process of Rusting.**—In spite of its occurrence for ages, it is only recently that the process of the formation of rust has been understood. It has long been known that oxygen and moisture are necessary for the rusting of iron, but the exact part played by each was not known. The two modern theories of the rusting of iron are the *Acid* and the *Electrolytic Theory*. According to the former the presence of an acid is necessary to the formation of rust, but even so weak an acid as carbonic may serve. The acid causes the metal to dissolve, and the oxygen changes the dissolved metal to rust, thereby liberating the acid, which is then capable of dissolving more metal, and so the process goes on. After several years of careful experimenting by different investigators, it now seems to be established that moisture and oxygen are sufficient for the continued rusting of iron, so that the electrolytic theory of rusting is the one more generally accepted.

The electrolytic theory postulates that when a metal dissolves it goes into solution as ions, i.e., atoms each carrying one or more (according to its valence) electric charges, and that it is only by giving up its charge that a metallic ion can escape from solution and become metal again. In the electrolytic cell this is merely visualizing Faraday's law, that the passage of a given quantity of current through an electrolyte is accompanied by the dissolving of a definite amount of material from the anode, and the deposition of a chemically equivalent amount at the cathode. All that the electrolytic theory of the solution of metals has done is to extend the operation of Faraday's law, previously confined, in men's minds, to electrolytic and voltaic cells, to every case of the dissolving of metals. Although no flow of current can be detected when a piece of iron is dissolving in acid, it is believed that the ions of metal still carry electric charges into the solution and that the hydrogen displaced from the acid also carries its normal electric charge, just as if the metal were caused to dissolve as anode and the hydrogen to be deposited at some cathode by the application of an electromotive force external to the cell.

\* *Jour. Iron and Steel Inst.*, 1907 (2), 103.

**847. The Function of Hydrogen in Corrosion.**—The statement is sometimes made that it is the hydrogen of acids that corrodes metals. It is evident that this is not so. The function of the hydrogen of acids in corrosion is merely to supply ions that are easily displaced from solution by metals, and which also escape from their surfaces and so do not prevent free access of the electrolyte, as is the case with many metallic ions, which, though more easily displaced than hydrogen, adhere more or less firmly to the surface of the metal on which they are deposited.

**848. The Function of Oxygen in Corrosion.**—So far there seems no need for oxygen in the electrolytic theory of the rusting of iron; but the dissolving of iron or any metal in an acid or any other electrolyte is (except where a change from a higher to a lower valence can occur, as with ferric salts) a case of displacement. Solution and deposition are inseparable; if deposition is prevented, the process of solution is also halted. It is here that oxygen comes into play as a deciding factor in corrosion. If two sheets of copper connected by a wire are suspended in a solution of copper sulphate, copper neither dissolves nor is deposited; the tendency toward solution or deposition at one plate is balanced by an equal tendency at the other. That solution shall occur it is necessary that a slight E.M.F. be applied to the system, when not only does copper dissolve from one sheet, but an equal weight of metal is deposited at the other.

In order that a metal shall dissolve in an electrolyte, e.g. hydrochloric acid, it is necessary that the forces tending toward solution and deposition shall be unequal. This idea has been stated in various ways: that the chemical affinity of the metal for the acid radical must exceed that of hydrogen for the same; that the potential of the metal must be greater than that of hydrogen; that the potential of the metal must be greater than the discharge potential of hydrogen on it; or that the solution pressure of the metal must exceed that of hydrogen. Not only does it require a greater driving force to displace hydrogen on one metal than on another, but the force or potential required increases with the amount of hydrogen already displaced and present on the surface of the metal. It follows then, that the dissolving of a metal may begin in some electrolyte from which it displaces hydrogen, but may be brought to a standstill before gas becomes visible and escapes, because the potential of the metal, or driving force, is no longer great enough to continue displacing hydrogen on a metal already highly charged with it. The only way in which dissolving of the metal can then continue is for a portion of the hydrogen to be removed by some means, so that the force required to deposit more of it may again be less than the potential of the metal. To do this is the function of oxygen in the rusting of iron, first clearly stated by W. H. Walker in 1907.\* “It was formerly thought that the action of oxy-

\* *Jour. Amer. Chem. Soc.*, 29, 1264.

gen as a factor necessary in corrosion was simply to oxidize the iron ions thrown into solution, and to precipitate them as rust. While it is true that this reaction does take place, and is indeed the most striking function that oxygen performs, it is really a secondary one, which is simply incidental to corrosion and not a necessary part of the action. Its real accelerative effect is due to the fact that it depolarizes the hydrogen which is set free by the reaction, and separates out on the metallic iron.\* Confirmation of this view of the function of oxygen in corrosion is seen in experiments on the corrosion of metals in acids,† in which the addition of oxidizing agents caused the rapid dissolving of several metals by acids which alone have little or no action on them.

**249. Conditions Affecting Corrosion.**—The corrosion of metals, including the rusting of iron and any other process which involves as a preliminary step the dissolving of a metal, requires an electrolyte. The rate of corrosion is determined by Ohm's law, i.e., it varies directly as the E.M.F. and inversely as the resistance. Other conditions being equal the lower the resistance of the electrolyte the more rapidly is the metal attacked. The rapid increase in the rate of corrosion observed with rise of temperature is due in part at least to the lessened resistance of electrolytes at high temperatures.

It is of course to be expected that a metal will not dissolve in an acid which forms an insoluble salt with it; why acids attack some metals readily, but not others, although the salts of both are soluble, may be understood by a study of the following table:

TABLE 1

Element.	Potentials * in Normal.		Discharge Potential of hydrogen on Metal.	Difference.
	Sulphate.	Chloride.		
Zn.....	0.524	0.503	Hg 0.548	Fe .....
Cd.....	0.162	0.174	Zn 0.468	Zn 0.056
Fe.....	0.093	0.087	Pb 0.408	Ni .000
Sn.....	.....	-0.085	Sn 0.298	Cd -0.083
Pb.....	.....	-0.095	Cd 0.248	Sn -0.383
H.....	-0.238	-0.249	As 0.152	Pb -0.503
Cu.....	-0.515	.....	Cu -0.002	Cu -0.516
As.....	.....	-0.550	Ni -0.022	As -0.708
Hg.....	-0.980	.....	Ag -0.082	Ag -0.882
Ag.....	-0.974	.....	Pt -0.112	Pt -1.028
Pt.....	.....	-1.140	Au -0.218	Au -1.144
Au.....	.....	-1.356	PtPt -0.238	Hg -1.528

\* Neuman, *Zeit. fur Phys. Chem.*, 1894; 14, 203.

\* W. H. Walker, *Jour. Iron and Steel Inst.*, 1909 (1), 69.

† *Trans. Amer. Electrochem. Soc.*, 1917, 32, 17.

Corrosion of a metal by acids and by solutions of the salts of other metals can be predicted from Table 1. Except as limited by the insolubility of its salt, a metal is corroded by solutions of all metals below it in potential. If the potential of a metal exceeds the discharge potential of hydrogen on it, this metal is readily corroded by acids whose salts of the metal are soluble; but if its potential is less than the discharge potential of hydrogen, acids dissolve it only at the rate at which the displaced hydrogen dissolves in the electrolyte, is removed by the oxidizing action of the air, or is otherwise disposed of. The special power possessed by nitric acid of dissolving lead, copper, silver, etc., whose potentials are less than the discharge potential of hydrogen on them, is due to its being an oxidizing agent as well as an acid.

The order and relative magnitude of the potentials of the metals not only foretells what metals will displace others from solution, but gives the order of chemical activity, and the stability of compounds of the different metals, those of higher potential being more active and forming more stable compounds. The discharge potential of hydrogen is the potential produced by the accumulation of hydrogen on the metal to such a degree that it begins to escape in visible bubbles. The column marked "difference" is the result of subtracting the discharge potential of hydrogen from the potential of the metal, and its magnitude should be an index of the rate of corrosion of the different metals by acids, provided no oxygen or oxidizing agent is allowed access to the metal. For example cadmium is very high in potential, and from this consideration alone might be expected to be corroded rapidly by acids, but the negative value of the "difference" indicates that it should dissolve in non-oxidizing acids only at the rate at which the displaced hydrogen is removed by the electrolyte; this accounts for its slow corrosion when used as a reference electrode in the 30 per cent sulphuric acid of a lead storage battery.

The protection of iron from rusting by immersion in a 5 per cent solution of sodium hydrate is predicted by the value  $-0.90$  for the "difference"; iron is incapable of displacing hydrogen from this solution, hence does not dissolve, and no rust can form. The effect upon the corrosion of iron of electro-negative impurities such as particles of graphite, bits of scale left on the surface, or the cementite present in steels, is not simply a matter of the initial E.M.F. between the iron and the other substance, but depends on the difference between the potential of iron and the discharge potential of hydrogen on these materials.

There are doubtless many incorrect values in the table, as most of the data is old, so that conclusions drawn from it can be relied on only in a broad and general sense.

The effect of amalgamation in preventing the dissolving of zinc by acids is due to the fact that the discharge potential of hydrogen on mer-

cury exceeds the potential of zinc; the remarkable diminution in the rate of attack of iron by sulphuric or hydrochloric acid, caused by the addition of a solution of arsenic, is due to a similar action—arsenic is precipitated on the surface of the iron and serves as cathode in the voltaic cell of which iron is anode; but the discharge potential of hydrogen on arsenic exceeds the potential of iron, hence iron is unable to displace hydrogen from the electrolyte except as an infinitesimal layer, and corrosion ceases.

**850. Rust a Stimulator of Corrosion.**—In 1849, R. Mallet \* said, "As every metal is positive to its own oxides, the adherent coat of rust upon iron, while it remains, powerfully promotes the corrosion of the metal beneath." Experiments on atmospheric corrosion of wrought iron, open hearth and Bessemer steels by Aston and Burgess † showed rust to have an accelerating effect in every case. In a later paper Aston ‡ ascribes the accelerating effect of rust on the corrosion of iron to its acting as a screen to prevent free access of air to the metal beneath it. "The underlying feature appears to be the relative access of oxygen to the surface of the electrodes. The electrolyte must reach both; then that to which oxygen has the more free access becomes the cathode, and the other is the anode. If two bare iron electrodes are separated by a partition of porous earthenware, parchment, etc., either may be made the cathode by bubbling air into this compartment, and not into the other. . . . Wet rust or a similar coating upon one electrode plays the role of a diaphragm permeable to the moisture, but preventing or slowing down the oxygen penetration." This theory of the function of rust seems adequate to account for the observed tendency for the corrosion of iron and steel to take the form of pits when it corrodes under such conditions that the first-formed rust is not continually removed. Pitting does not occur in the corrosion of iron or steel by acids or as anode in the refining of iron electrolytically, but the corrosion of these materials almost invariably takes this form when they are buried in the ground or immersed in a stagnant, neutral electrolyte, so that the patches of rust first formed can adhere. Whether steel corrodes uniformly or in pits, is then determined, not by the nature of the steel, but by its surroundings. Given uniform surroundings corrosion will be uniform; with freer access of the depolarizing air to some spots than to others pitting results.

**851. Concentration Cells and Thermal E.M.F. May Cause Corrosion.**—It has long been known that an E.M.F. exists between two pieces of the same metal immersed in an electrolyte that differs in concentration at the electrodes; this constitutes a "concentration cell," and although

\* *Report of Brit. Assoc. for Adv. of Sci.*, 1849, p. 111.

† *Trans. Amer. Electrochem. Soc.*, 1912, 22, 233.

‡ *Trans. Amer. Electrochem. Soc.*, 1916, 29, 449.

the E.M.F. is small, its continued action in good-conducting electrolytes is often responsible for serious corrosion.

Burgess and Engle\* have shown that an E.M.F. is produced between two pieces of iron in an electrolyte when one of them is heated to a higher temperature than the other, and have suggested that this may explain many cases of corrosion of the tubes of locomotive boilers. Several years ago a severe case of corrosion occurred in the steel water-jacket of a copper blast-furnace at Douglas, Ariz.,† the inner plates of which were deeply corroded, while the outer plates and stiffeners were unattacked. The absence of acidity in the water, and the entire freedom from corrosion of boilers using the same water were puzzling features of the case. The corrosion can be fully accounted for by the existence of an E.M.F. between the highly heated inner shell as anode and the cooler outer part as cathode, the water, which was found to contain much sodium and potassium as sulphate, chloride, and carbonate, furnishing the electrolyte. On account of its alkalinity the water could not attack iron except the metal was anode, and hence did not corrode the boilers, where the E.M.F. was lacking, or at least was very much smaller due to more uniform heating.

**852. Effect of Stress and Strain on Corrosion.**—Several investigators have studied the effects of stress and strain on the potential and corrodibility of iron and steel,‡ as a result of which it may be concluded that the cold-working of steel or iron raises its potential and increases the rate at which it is corroded by acids. Fig. 1 shows results obtained by Thickens in corroding, in N/4 hydrochloric acid, cylinders of iron and steel that had been strained by torsion or in tension, A and B being mild steel, C, electrolytic iron, and D a cylinder of mild steel that was stretched until necking down occurred, when it was machined to a true cylinder and suspended in the acid. Fig. 2 shows the effect of corrosion by acid on a punched plate of half-inch steel; not only was there severe corrosion of the strained metal at the hole, but curious lines of strain are seen to extend to a distance of an inch from the hole.

With regard to stresses which do not produce permanent distortion of the metal, i.e., which are within the elastic limit, the results of different experimenters are so conflicting that this question must be regarded as still undecided.

**853. Puzzling Corrosion of Turbine-driven Propellers.**§—With the

\* *Trans. Amer. Electrochem. Soc.*, 1908, 13, 37.

† *Trans. Amer. Inst. Min. Eng.*, 1908, 38, 877.

‡ *Warms, Bull. U. S. Geol. Survey*, No. 94, 48-73; *Andrews, Proc. Inst. Nav. Eng.*, 1894, 118, 356; *Hambuechen, Bull. Univ. Wis.*, No. 42; *Richards and Behr, Pub. Carnegie Inst.*, 1906; *Walker and Dill, Trans. Amer. Electrochem. Soc.*, 11, 153; *Thickens Thesis, Univ. Wis.*, 1908; *Trans. Amer. Electrochem. Soc.*, 1908, 13, 7.

§ *The Engineer*, 1908, 105, 535, 539; 1909, 107, 397; 1910, 110, 252. *Engineering*, 1912, 93, 33, 687, 884; 1913, 96, 690, 726, 761; 1914, 97, 535.

general adoption of bronze instead of cast iron or steel for the propellers of steamships it seemed as if the former troubles from corrosion were forever ended, and that the propeller might be regarded as a permanent piece of equipment. In 1908, however, *The Engineer* announced a new variety of corrosion which, in a few weeks or even days, destroyed the best bronze propellers. It consisted of pitting to the depth of three-quarters of an inch or more over an area of 40 to 50 square inches on the driving or astern face of the wheel.

This peculiarly destructive form of corrosion was first noticed on the propellers of ocean-going destroyers of the British navy, and, curiously, was confined to vessels driven by turbine engines. Corrosion was so

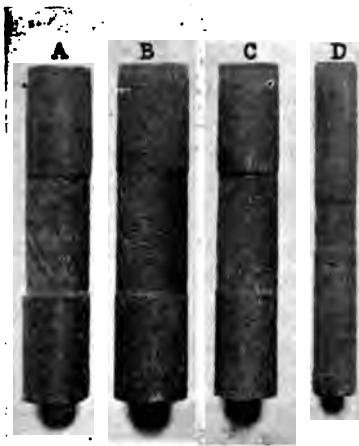


FIG. 1.



FIG. 2.

serious that a propeller was often ruined during the trials, and before the vessel was put into service. Later it was found that this form of corrosion was also taking place, but much more slowly, on the *Mauretania*, *Lusitania*, and a few other "liners." In the interesting and lengthy discussion which followed the cause was assigned to faulty material, segregation of impurities, oxidation of the metal by air drawn from the water by "cavitation," erosion by the water due to the high speed of turbine engines, and to electrolytic action between strained and unstrained metal. A study of the examples shown in the several articles leads to the conclusion that the true cause is that last mentioned, and that corrosion may be prevented by (a) lessening the power transmitted by a single wheel, (b) the substitution of a stronger material for that used, (c) increasing the thickness of the wheel.

In many cases bending of the blades was found to have occurred at the places corroded. Corrosion was the result of a rise in potential of



the metal caused by cold-working, and illustrates the serious damage that may be caused by an exceedingly small E.M.F., provided a good electrolyte is continually present. It is still an open question whether or not a portion of the observed corrosion is due to that bending of the blades within the elastic limit, which must occur while the vessel is running, but which does not result in permanent distortion of the propeller.

These examples of corrosion induced by strains indicate the desirability of annealing all metals and alloys which are likely to be subjected to corrosive influences.

**854. Effect of Various Elements on the Corrodibility of Iron and Steel.**—From the nature of the constitution of alloys it is evidently impossible from a mere knowledge of the resistance to corrosion of the alloying elements to predict correctly regarding corrosion of alloys, either by acids or on exposure to the atmosphere. Corrosion will be affected, not only by the chemical activity or resistance of the element added, but by the state in which this exists in the alloy. Whether the alloying element unites with the original metal forming a compound, forms a solid solution with it, or separates in the elemental state, are quite as important factors in determining corrosion as is the chemical nature of the alloying element itself. The quantity of the alloying element added will also influence profoundly the corrodibility of the alloy; if the added element forms with the original metal a compound which is strongly resistant to corrosion, and on which the discharge potential of hydrogen is less than on the original metal, its addition in small amounts will accelerate corrosion by forming active local couples; but as the amount added is continually increased a point is finally reached at which the alloy consists entirely of the compound, when the resistance of the alloy to corrosion will be far greater than that of the original metal.

When the constitution of alloys is thoroughly known and their corrosion has been more fully studied, it will probably be possible to predict correctly the corrodibility of many alloys before making them; but at present experiment is the only safe guide regarding the corrosion of most new alloys.

Heat treatment is an important factor in the corrodibility of alloys, for besides removing differences of potential due to strain, it often produces changes in the constituents of the alloy.

The effect on corrosion of adding other elements to iron has been extensively studied, but lack of space permits mention of the effects of only a few of the more common ingredients of steel.

**Carbon.**—Corrosion of annealed steel in water and dilute sulphuric acid rises with the carbon content to a maximum at the eutectic point (89 per cent C). Quenched and tempered steels show a continuous

rise in corrosion with increase of carbon up to 0.96 per cent.\* The temperature to which a quenched steel is reheated has a marked effect on its corrosion in 1 per cent sulphuric acid,† a steel containing 0.95 per cent carbon showing a sharp rise in solubility when reheated to 400° C., amounting to six times the rate of attack on untempered steel.

*Copper.*—The remarkable lessening of the corrosion of iron by the atmosphere, water, sea water, and acids, caused by the addition of very small amounts of copper, has been the subject of many investigations.‡ Although resistance to corrosion increases with the copper content up to 2 to 3 per cent of copper, for resisting atmospheric corrosion there appears to be no advantage in exceeding 0.25 per cent of copper, and quantities as small as 0.05 per cent are said to have a marked influence on the rate of corrosion of iron.

*Manganese.*—It is generally accepted that manganese in steel causes increased corrosion. Hadfield and Friend § found 0.7 per cent of manganese to cause a great increase in the rate of corrosion of carbon steels in tap water and in artificial sea water, but above 2 per cent of manganese corrosion is much decreased. In 0.1 and 0.5 per cent sulphuric acid corrosion increases with the manganese content up to 12 per cent manganese, the highest manganese used.

*Oxygen.*—From theoretical considerations combined oxygen would be expected to accelerate the corrosion of iron, and Law || corroborates this by observations on rusting and corrosion in dilute acid.

*Silicon.*—Silicon-iron alloys containing about 15 per cent of the former element are now extensively used under various trade names for the construction of chemical apparatus, although their hardness and brittleness are a serious drawback to their usefulness. Up to 20 per cent of silicon, iron silicide ( $\text{Fe}_2\text{Si}$ ) forms a solid solution with iron, which fact, in connection with the small weight of silicon needed to produce a large quantity of the highly resistant iron silicide, accounts for the great resistance to acids secured by the addition of only a moderate amount of silicon, in comparison with additions of other elements for protection.

*Sulphur.*—"Sulphur is universally regarded as a stimulator of corrosion." ¶

\* Chapelle, *Jour. Iron and Steel Inst.*, 1912 (1), 270.

† Hadfield and Friend, *Jour. Iron and Steel Inst.*, 1916 (1), 48.

‡ F. H. Williams, *Proc. Eng. Soc. West Penn.*, 1900, 16, 23; Stead and Wigham, *Jour. Iron and Steel Inst.*, 1901 (2), 122; P. Breuil, *Jour. Iron and Steel Inst.*, 1907 (2), 1; Burgess and Aston, *Trans. Amer. Electrochem. Soc.*, 1912, 22, 244; D. M. Buck, *Jour. Ind. and Eng. Chem.*, 1913, 5, 447; *Jour. Ind. and Eng. Chem.*, 1916, 8, 209.

§ *Jour. Iron and Steel Inst.*, 1916 (1), 48.

|| *Jour. Iron and Steel Inst.*, 1907 (2), 103.

¶ Friend, *Corrosion of Iron and Steel*, p. 321.

**855. Protection of Iron from Rusting.**—It is evident from the electrolytic nature of corrosion that the rusting of iron may be prevented by keeping it from contact with electrolytes, by rendering the iron "passive," or by making it cathode, the last being accomplished by contact with a more positive metal such as zinc, or by the application of an external E.M.F.

The first method is carried out by painting, enameling, covering with another metal, or forming on the surface of the iron by chemical action a coat of oxide or other compound. For success by this method it is necessary that the coating be impervious to moisture, and remain unbroken. These are difficult conditions to fulfill, particularly when the object is exposed out of doors and subjected to wear. Paints soon lose their insulating power, and are readily scratched, enamels become chipped, electro-plate usually contains thin spots which soon wear through, and the compounds formed in the Bower-Barff and other similar processes must be kept so thin in order to prevent chipping and peeling, that their serviceability is less than if they could be given a considerable thickness. The subject of rust-resisting paints is receiving much attention from the technical societies interested, and marked improvements are to be expected. At first sight it might seem that coating iron with a less corrodible metal, either by dipping it in the melted metal or by electro-plating, should be a perfect remedy for rusting; but experience has shown that of all the metals so applied to iron, only zinc or cadmium affords satisfactory protection, and when exposed to a good electrolyte like sea water even zinc fails to protect for more than a year or two. In coating by dipping, the thickness of the coating is limited, and only metals and alloys of low melting-point can be used. Galvanized iron, tin plate, and terne plate, the latter consisting of iron coated with an alloy of lead and tin, are made in enormous quantities by this method, but the durability of the last two is hampered by the corrosion induced by "local action" between the coating and the iron whenever the latter becomes exposed. In protecting iron by electro-plating there is a much wider choice of materials than in the dipping process, and theoretically the coating may be made of any thickness desired, but in practice its thickness is restricted by the increasing roughness of thick deposits and their liability to peel. For these reasons, coupled with the cost of producing heavy deposits, commercial electro-plate is usually thinner than the heaviest coatings made by dipping.

**856. Utilization of Passivity to Prevent Rusting.**—Kier in 1790 observed that iron, after treatment with fuming nitric acid, had lost its power of precipitating silver from a solution of silver nitrate, in other words, that the iron had been rendered immune to corrosion by silver nitrate. Chromic acid and several other oxidizing agents have since been found to

exercise a similar effect in preventing iron from reacting as usual toward many reagents. Attempts have been made to prevent rusting by rendering iron passive, but, unfortunately, passivity soon ceases after removal of the iron from the passivating solution, and the presence of certain salts may entirely prevent passivity; for these reasons the utilization of passivity for preventing the rusting of iron is as yet very limited. Although small amounts of alkalies accelerate the corrosion of iron, a strong solution entirely prevents rusting, and this passivity of iron in alkaline solutions has been very successfully applied to the prevention of corrosion in boilers.\* Certain pigments used in paint, e.g. the chromates, are often classed as "inhibitors" from their supposed effect in preventing rusting by rendering iron passive.

**857. Protection by Contact with Zinc.**—The protection of metals from corrosion by placing a piece of zinc in contact with them seems to have been discovered by H. Davy † in 1824, and was first applied by him to prevent the corrosion of the copper sheathing of wooden ships. Since that time protection by contact with zinc has been applied to boilers and marine condensers, and plates of zinc are usually attached to the hull of steel vessels in the vicinity of bronze propellers, sea cocks, etc., which otherwise would induce serious corrosion of the steel in immediate contact with them.

The principle involved is that corrosion of a metal is lessened by the passage of current to it as a cathode, but there are two difficulties to be overcome in applying this method; first, a good electrical connection must be maintained between the two metals; and second, in some electrolytes a coating forms on a zinc anode which may lower its potential to such an extent that the protective action is entirely nullified. For the protection of copper and brass in sea water iron appears to be a better contact metal than zinc, in spite of the higher initial potential of the latter.

**858. Prevention of Corrosion by Current from a Dynamo.**—In the generation of current for plating the corrosion of zinc in primary cells long ago gave way to the dynamo, and in the prevention of corrosion in large boiler installations by cathodic action, voltaic action has recently been abandoned in favor of the dynamo. Wherever corrosion is severe such protection results in a very considerable saving in the cost of repairs, less interruptions in service, and a higher efficiency of boilers from prevention of the formation of scale. The method seems destined to a wide usefulness.

**859. Corrosion of Non-ferrous Metals.**—The observed behavior of the non-ferrous metals in regard to corrosion is in fairly good agreement

\* F. Lyon, *Jour. Amer. Soc. Nav. Eng.*, 1912, 24, 845.

† *Phil. Trans. Royal Soc.*, 114, 151.

with the principles set forth on page 23, modified by the formation of insoluble coatings upon them. The highly positive metals, aluminum and zinc, owe their durability entirely to the formation of an insoluble compound on their surfaces, and although durable in air, dissolve in all electrolytes which remove the protective coatings. Because of the high potentials of these metals, purity and freedom from "local couples" are especially important. Aluminum is corroded rapidly by the halogen acids, seriously by their salts, and vigorously by alkalis. Lead, tin, copper, brass, bronze, Munz metal, Monel, and silicon-iron alloys are the materials most depended on to withstand severe corrosive conditions when platinum cannot be used on account of cost, as in sea water, for conveying corrosive liquids, etc. Exclusion of oxygen, annealing to remove strains, and freedom from contact with more negative electrical conductors are highly important in minimizing corrosion. In neutral electrolytes brass is subject to the peculiar form of corrosion known as "dezincification," in which the zinc dissolves, leaving a skeleton of porous copper that is utterly lacking in mechanical strength.

**830. Corrosion by Stray Currents.**—In addition to the chemical or natural corrosion to which metals buried in the earth are subject, any continuous length of metal, such as a water or gas main, or the lead sheath of telephone cables, may suffer from electrolytic corrosion. This may be so serious as to destroy in a single year a cast-iron water main that should normally have a life of 50 to 100 years, while the thinner service pipes may last only a few months in especially bad situations. This damage results from the practice of using the rails for returning to the power station the electric current used for operating street railways. A greater or less proportion of the total current flows from the rails through the soil to the pipes, follows the latter to the vicinity of the power station, where it returns to the rails once more in order to reach the negative terminal of the dynamo.

Whenever "direct current" leaves a metallic anode to enter an electrolyte the metal is liable to be corroded, the extent of such corrosion depending on the same principles which control the corrosion of anodes in the electro-plating and refining of metals. The most important factors in the corrosion of anodes are: the chemical nature of the particular metal or alloy which serves as anode, the amount of current flowing, the time, the nature, amount, and concentration of salts in the soil, the temperature, current density, and rate of circulation of the electrolyte. With alternating current corrosion is usually negligible in comparison with that produced by direct current. Faraday's and Ohm's laws are of fundamental importance in electrolytic corrosion. According to the former the amount of corrosion should be proportional to the current which leaves the anode, to the time, and to the chemical equivalent of the metal; the

amount of current being determined by Ohm's law, that the current equals the electromotive force divided by the resistance.

Current which enters the system of pipes at different points throughout a city will leave the pipes in the vicinity of the power station. Two electrolytic cells are thus formed by the rails, earth, and pipes; one where current enters the pipes, in which the rails are anodes, and the other near the station, in which the pipes act as anodes, and are corroded. Although nothing is made public concerning corrosion of the rails, it is evident that this must occur to about the same extent as corrosion of the pipes, but will be distributed over a much larger area of the city. Soil conducts electricity only as an electrolyte, and by virtue of solutions contained in it, so that the resistivity of different soils varies greatly, as does that of the same soil under different conditions of moisture. Temperature is an important factor in the resistance of electrolytes, and when the ground is frozen to the depth of a few feet corrosion of the pipes by stray currents is at a standstill.

**861. The Danger District.**—This is the portion of the pipe system from which current flows into the earth on its way to the rails, and it may be located by taking measurements with a voltmeter between the rails and hydrants. Wherever the pipes are positive to the rails the former are in danger of corrosion; but a high voltmeter reading at one place does not always mean a greater flow of current than a smaller reading elsewhere, for the voltmeter reads only the IR drop produced by flow of current in the earth, and there is the possibility that in a soil of high resistivity a small current may produce a greater fall of potential than is caused by a larger current which flows in a better-conducting soil.

**862. Extent of Corrosion.**—The amount of current flowing in a single pipe has been found to vary from a few amperes to several hundred, and in the case of "bonded pipes" in some of the largest cities, to several thousand amperes. Faraday's law predicts the dissolving of 15 pounds of iron by one ampere flowing from the metal 18 hours a day for a year, which leads one to wonder, not that damage to the extent of hundreds of thousands of dollars is caused annually by stray currents, but that any pipes remain undestroyed. The discrepancy between the amount of current known to be carried by pipes, and the relatively slow rate of their destruction, has given rise to much controversy and misunderstanding. The dissolving of 15 pounds of iron per ampere-year is for a current efficiency of 100 per cent, and requires that the entire current be spent in dissolving iron, and none of it employed in liberation of oxygen at the anode. Experience with plating and refining solutions has shown that the current efficiency of anode corrosion is greatly affected by the current density (number of amperes per sq. foot of anode surface), by the concentration of the electrolyte, and its rate of circulation,—low current

density, high concentration, and rapid circulation tending toward a high efficiency, and vice versa. While the exact current density and the concentration of the electrolyte is unknown in the corrosion of underground pipes, the extreme dilution and stagnation of the average electrolyte in the soil of city streets should lead to a low efficiency of anode corrosion, except where the bad practice of applying salt to switches to thaw ice has been indulged in.

**863. Corrosion at Low Voltage.**—The statement is often made that the voltage between pipes and rails must exceed the E.M.F. of decomposition of water (1.7 volts) before corrosion can occur. This is an error. It is only with insoluble electrodes, if both are of the same metal, that this voltage is necessary for the passage of current; with a soluble anode any voltage, however small, will cause some current to flow. No fixed value can be given for polarization (the counter E.M.F. caused by products of electrolysis) that will apply to stray-current corrosion in general; this varies from 0.01 to 2.0 volts, according to the current density, and the material, concentration, and freedom to circulate of the electrolyte.—factors which are difficult to determine in electrolysis of pipes buried underground.

**864. Joint Electrolysis.**—A peculiar form of corrosion sometimes occurs in water pipes, known as joint electrolysis. The average lead joint in water mains has a very low resistance, so that no appreciable current is driven from the pipe at the joint; but when a joint of high resistance is encountered a considerable proportion of the current may pass from one section of pipe to the next by way of the soil instead of through the packing of the joint, causing corrosion on the anode side of the joint. Since this process is repeated at every bad joint, the damage done by a definite amount of current may be many times greater than in ordinary stray-current corrosion. Fortunately high-resistance joints are rather rare.

**865. Remedies.**—Because of lack of space it is only possible to mention some of the more important remedies that have been used or proposed for prevention of stray-current corrosion:

1. Lowering the voltage drop on the track by:

- (a) Better bonds.

- (b) Use of copper cables in parallel with the rails.

- (c) Insulated negative feeders attached to the track at suitable points.

- (d) Negative "boosters" to draw off current at desired points on the track.

2. Bonding of pipes near the station to the rails or to the negative terminal of the dynamo, so that current leaves the pipes by a metallic instead of an electrolytic conductor.

3. Insulation of the pipes from the earth.
4. Insulation at pipe joints.
5. Use of the double trolley, i.e. carrying the return current on an insulated overhead wire.
6. The use of alternating instead of direct current for the operation of street cars.



parties shall unite in the selection of a disinterested person to select the samples, and both parties shall be bound by the results of samples thus selected.

3. *Number of Samples per Lot.*—In general, one sample of ten bricks shall be tested for every 10,000 bricks contained in the lot under consideration; but where the quantity exceeds 100,000, the number of samples tested may be fewer than one 10,000, provided that they shall be distributed as uniformly as practicable over the entire lot.

4. *Shipment of Samples.*—Samples which must be transported long distances by freight or express shall be carefully put up in packages holding not more than two bricks each. When more than six bricks are shipped in one package, it shall be arranged as to carry two parallel rows of bricks side by side, and these rows shall be separated by a partition. In event of some of the bricks being cracked or broken in transit, the sample shall be disqualified if there are not remaining ten sound undamaged bricks.

5. *Storage and Care of Samples.*—Samples shall be carefully handled to avoid breakage or injury. They shall be kept in the dry so far as practicable. If wet when received, or known to have been immersed or subjected to recent prolonged wetting, they shall be dried for at least six hours in a temperature of 100° F. before testing.

#### THE CONSTRUCTION OF THE RATTLER

6. *General Design.*—The machine shall be of good mechanical construction, compact, and shall conform to the following details of material and dimensions, and shall consist of barrel, frame, and driving mechanism as herein described. Accompanying these specifications is a complete drawing of a rattler which will meet the requirements, and to which reference should be made (Plate I).

7. *The Barrel.*—The barrel of the machine shall be made up of the heads, headliners, staves and stave-liners.

The heads may be cast in one piece with the trunnions, which shall be 2½ in. diameter, and shall have a bearing 6 in. in length, or they may be cast with heavy hubs which shall be bored out for 2½-in. shafts, and shall be keyseated for two keys, each ½ by ½ in. and spaced 90 degrees apart. The shaft shall be a snug fit and when keyed shall be entirely free from lost motion. The distance from the end of the shaft to the trunnion to the inside face of the head shall be 15½ in. in the head for the driving end of the rattler, and 11½ in. for the other head, and the distance from the face of the hub to the inside face of the heads shall be 5½ in.

The heads shall be not less than ½ in. thick, nor more than ¾ in. thick. In each head shall be a regular 14-sided polygon inscribed in a circle 28½ in. in diameter. Each head shall be provided with flanges not less than ½ in. thick and extending outward 2½ in. from the inside face of the head to afford a means of fastening the staves. The surface of the flanges of the head shall be smooth and give a true and uniform bearing for the staves. To secure the desired true and uniform bearing the surfaces of the flanges of the head shall be either ground or machined. The flanges shall be slotted on the outer edge, so as to provide for two ½-in. bolts at each end of each stave, said slots to be ½ in. wide and 2½ in., center to center. Each slot shall be provided with a recess for the bolt head, which shall act to prevent the turning of the same. Between each two slots there shall be a brace ½ in. thick, extending down the outward side of the head not less than 2 in.

There shall be for each head a cast-iron headliner 1 in. in thickness and conforming to the outline of the head, but inscribed in a circle 28½ in. in diameter. This headliner shall be fastened to the head by seven ½-in. cap-screws, through the head from the

parties shall unite in the selection of a disinterested person both parties shall be bound by the results of samples thus taken.

3. *Number of Samples per Lot.*—In general, one sample for every 10,000 bricks contained in the lot under consideration. If the quantity exceeds 100,000, the number of samples tested shall be 10,000, provided that they shall be distributed as uniform throughout the entire lot.

4. *Shipment of Samples.*—Samples which must be taken for freight or express shall be carefully put up in packages of 10 bricks each. When more than six bricks are shipped in a package, they shall be arranged as to carry two parallel rows of bricks side by side, separated by a partition. In event of some of the bricks being broken in transit, the sample shall be disqualified if there are not ten good bricks.

5. *Storage and Care of Samples.*—Samples shall be kept in a dry place free from breakage or injury. They shall be kept in the dry so far as possible, or known to have been immersed or subjected to moisture, they shall be dried for at least six hours in a temperature of 100° F.

#### THE CONSTRUCTION OF THE RATTLER

6. *General Design.*—The machine shall be of good design, and shall conform to the following details of construction. It shall consist of barrel, frame, and driving mechanism as hereinafter specified. Accompanying these specifications is a complete drawing of a rattler, and to which reference should be made (Plate I).

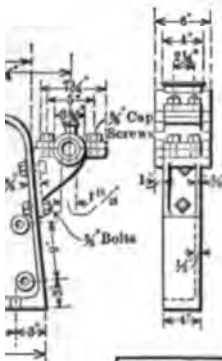
7. *The Barrel.*—The barrel of the machine shall be made of cast-iron liners, staves and stave-liners.

The heads may be cast in one piece with the trunnion, and shall have a bearing 6 in. in length, or they may be made in two pieces, which shall be bored out for 2½-in. shafts, and shall be 1½ in. by ½ in. and spaced 90 degrees apart. The shaft shall be entirely free from lost motion. The distance from the trunnion to the inside face of the head shall be 15½ in. in the case of the rattler, and 11½ in. for the other head, and the distance from the inside face of the heads shall be 5½ in.

The heads shall be not less than ½ in. thick, nor more than ¾ in. Each head shall be a regular 14-sided polygon inscribed in a circle 28½ in. in diameter. Each head shall be provided with flanges not less than ½ in. thick, and 2½ in. from the inside face of the head to afford a means of securing the staves. The surface of the flanges of the head shall be smooth and give a true bearing for the staves. To secure the desired true and uniform bearing of the head shall be either ground or machined. On the outer edge, so as to provide for two ½-in. bolts at right angles to the face of the head, there shall be two slots to be ½ in. wide and 2½ in., center to center. Each slot shall be 1½ in. deep, and shall act to prevent the turning of the head. Each two slots there shall be a brace ½ in. thick, extending across the head not less than 2 in.

There shall be for each head a cast-iron headliner 1 in. thick, and shall fit the outline of the head, but inscribed in a circle 28½ in. in diameter. The liner shall be fastened to the head by seven ½-in. cap-screws.





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headliners become worn down  $\frac{1}{8}$  in. below their initial surface, they shall be replaced with new ones. The shall be hard machinery iron and should contain not less than carbon.

ade of 6-in. medium-steel structural channels,  $27\frac{1}{2}$  in. long near foot. The staves shall have two holes  $\frac{1}{4}$  in. in diameter, center line of the holes being 1 in. from the end and  $1\frac{1}{2}$  in. either center line. The spaces between the staves shall be as uniform and not exceed  $\frac{1}{8}$  in.

Each of each stave shall be protected by a liner  $\frac{1}{2}$  in. thick by plate. The liner shall consist of medium-steel plate, and shall be three  $\frac{1}{2}$ -in. rivets, one of which shall be on the center line both the longitudinal center line and spaced 7 in. from the center shall be countersunk on the face of the liner and the rivets pipped off flush with the surface of the liners. These liners to time, and if found loose shall be at once re-riveted.

When of which a stave-liner is found detached from the stave shall be rejected. When a new rattler, in which a complete charge, is first put into operation, it shall be charged with 400 lb. of shot in the same proportions as provided in Section 9, and shall be fired at the usual prescribed rate of speed. The shot shall be standard shot charge inserted, after which the rattler may be used.

For more than seventy consecutive tests without renewing staves shall be removed and relined at a time in such a way that upon one side of the barrel and the other upon the opposite side staves changed shall be consecutive but not contiguous; for example, 5 and 12, 7 and 14, 2 and 9, 4 and 11, 6 and 13, etc., to the end of the barrel at all times shall present the same relative condition. The staves should be made at the time when the shot charges are fired. The record must show the numbers of charges run since the stave was placed in position.

The heads shall form a barrel 20 in. long, inside measurements. The liners of the staves shall be so placed as to drop between the staves shall be bolted tightly to the heads by four  $\frac{1}{2}$ -in. bolts provided with a lock nut, and shall be inspected at not less than every fifth test and all nuts kept tight. A record shall be made of the condition in what condition the bolts were found.

**Driving Mechanism.**—The barrel shall be mounted on a casting of sufficient strength and rigidity to support it without undue vibration. The barrel shall be mounted with or without the interposition of wooden plates, and shall be bolted by bolts at not less than four points.

The driving pinion whose ratio of driver to driven is not less than one to two on which the driving pinion is mounted shall not be less than 6 in. in length. If a belt drive is used, the belt shall be not less than 18 in. in diameter and  $6\frac{1}{2}$  in. in face. A belt at all times adjusted, to avoid unnecessary slipping, should be used.

The abrasive charge shall consist of cast-iron spheres. The larger spheres shall be 3.75 in. in diameter and shall weigh 1.5 lb. each. Ten spheres of this size shall be used.

Separately after each ten tests, and if the weight of any large

sphere falls to 7 lb. (3.175 kg.) it shall be discarded and a new one substituted; provided, however, that all of the large spheres shall not be discarded and substituted by new ones at any single time, and that so far as possible the large spheres shall compose a graduated series in various stages of wear.

When new, the smaller spheres shall be 1.875 in. in diameter and shall weigh approximately 0.95 lb. (0.43 kg.) each. In general, the number of small spheres in a charge shall not fall below 245 nor exceed 260. The collective weight of the large and small spheres shall be as nearly 300 lb. as possible. No small sphere shall be retained in use after it has been worn down so that it will pass a circular hole 1.75 in. in diameter, drilled in an iron plate  $\frac{1}{4}$  in. in thickness, or weigh less than 0.75 lb. (0.34 kg.). Further, the small spheres shall be tested, by passing them over the above plate or by weighing, after every ten tests, and any which pass through or fall below the specified weight, shall be replaced by new spheres; provided, further, that all of the small spheres shall not be rejected and replaced by new ones at any one time, and that so far as possible the small spheres shall compose a graduated series in various stages of wear. At any time that any sphere is found to be broken or defective it shall at once be replaced.

The iron composing these spheres shall have a chemical composition within the following limits:

Combined carbon .....	Not under 2.50 per cent
Graphitic carbon.....	“ over 0.25 “
Silicon.....	“ “ 1.00 “
Manganese.....	“ “ 0.50 “
Phosphorus.....	“ “ 0.25 “
Sulphur.....	“ “ 0.08 “

For each new batch of spheres used, the chemical analysis shall be furnished by the maker or be obtained by the user, before introducing into the charge, and unless the analysis meets the above specifications, the batch of spheres shall be rejected.

#### THE OPERATION OF THE TEST

10. *The Brick Charge.*—The number of bricks per test shall be ten for all bricks of so-called “block-size,” whose dimensions fall between 8 and 9 in. in length, 3 and  $3\frac{1}{4}$  in. in breadth, and  $3\frac{1}{4}$  and  $4\frac{1}{4}$  in. in thickness.\* No brick should be selected as part of a regular test that would be rejected by any other requirements of the specifications under which the purchase is made.

11. *Speed and Duration of Revolution.*—The rattler shall be rotated at a uniform rate of not less than 29.5 nor more than 30.5 revolutions per minute, and 1800 revolutions shall constitute the test. A counting machine shall be attached to the rattler for counting the revolutions. A margin not to exceed ten revolutions will be allowed for stopping. Only one start and stop per test is generally acceptable. If, from accidental causes, the rattler is stopped and started more than once during a test, and the loss exceeds the maximum permissible under the specifications, the test shall be disqualified and another made.

12. *The Scales.*—The scales must have a capacity of not less than 300 lb., and must be sensitive to 0.5 oz., and must be tested by a standard test weight at intervals of not less than every ten tests.

\* Where brick of larger or smaller sizes than the dimensions given above for blocks are to be tested, the same number of bricks per charge should be used, but allowance for the difference in size should be made in setting the limits for average and maximum rattler loss.

13. *The Results.*—The loss shall be calculated in percentage of the initial weight of the brick composing the charge. In weighing the rattled brick, any piece weighing less than 1 lb. shall be rejected.

14. *The Records.*—A complete and continuous record shall be kept of the operation of all rattlers working under these specifications. This record shall contain the following data concerning each test made:

1. The name of the person, firm or corporation furnishing each sample tested.
2. The name of the maker of the brick represented in each sample tested.
3. The name of the street, or contract, which the sample represented.
4. The brands or marks upon the bricks by which they were identified.
5. The number of bricks furnished.
6. The date on which they were received for test.
7. The date on which they were tested.
8. The drying treatment given before testing, if any.
9. The length, breadth and thickness of the bricks.
10. The collective weight of the ten large spherical shot used in making the test at the time of their last standardization.
11. The number and collective weight of the small spherical shot used in making the test, at the time of their last standardization.
12. The total weight of the shot charge, after its last standardization.
13. Certificate of the operator that he examined the condition of the machine as to staves, liners, and any other parts affecting the barrel, and found them right at the beginning of the test.
14. Certificate of the operator of the number of charges tested since the last standardization of shot charge and last renewals of stave liners.
15. The time of the beginning and ending of each test, and the number of revolutions made by the barrel during the test, as shown by the indicator.
16. Certificate of the operator as to number of stops and starts made in each test.
17. The initial collective weight of the ten bricks composing the charge and their collective weight after rattling.
18. The loss calculated in percentage of the initial weight; and the calculation itself.
19. The number of broken bricks and remarks upon the portions which were included in the final weighing.
20. General remarks upon the test and any irregularities occurring in its execution.
21. The date upon which the test was made.
22. The location of the rattler and name of the owner, upon which the test was made.
23. The certificate of the operator that the test was made under the specifications of the American Society for Testing Materials and that the record is a true record.
24. The signature of the operator or person responsible for the test.
25. The serial number of the test.

In the event of more than one copy of the record of any test being required, they may be furnished on separate sheets, and marked duplicates, but the original record shall always be preserved intact and complete.

For the convenience of the public, the accompanying blank form, which provides space for the necessary data, is furnished and its use recommended.

Serial No. ....

## REPORT OF STANDARD RATTLER TEST OF PAVING BRICK

## IDENTIFICATION DATA

Name of the firm furnishing sample. ....  
 Name of the firm manufacturing sample. ....  
 Street or job which sample represents. ....  
 Brands or marks on the brick. ....  
 Quantity furnished. .... Drying treatment. ....  
 Date received. .... Date tested. ....  
 Length. .... Breadth. .... Thickness. ....

## STANDARDIZATION DATA

Weight of Charge. (After Standardization)	Condition of Locknuts on Staves.	Condition of Seal s.	Number and Position of Fresh Stave Liners.	Repairs. (Note any re- pairs affecting the con- dition of the barrel.)
10 Large spheres				
Small spheres..				
Total. ....				

Number of charges tested since last inspection. ....

## RUNNING DATA

	TIME READING.			Revolution Counter Readings.	Running Notes, Stops, etc.
	Hours	Minutes.	Seconds.		
Beginning of test					
Final reading					

## WEIGHTS AND CALCULATIONS

		Percentage Loss. (Note.—The Calculation Must Appear.)
Initial weight of ten bricks. ....		
Final weight of same. ....		
Loss of weight .....		

Number of broken bricks and remarks on same. ....

I certify that the foregoing test was made under the specifications of the American  
 Society for Testing Materials, and is a true record.

(Signature of tester).....

Date.....

Location of laboratory.....



## ACCEPTANCE AND REJECTION OF MATERIAL

15. *Basis of Acceptance or Rejection.*—Paving bricks shall not be judged for acceptance or rejection by the results of individual tests, but by the average of no less than five tests. Where a lot of bricks fail to meet the required average, it shall be optional with the buyer whether the bricks shall be definitely rejected or whether they may be regraded and a portion selected for further test as provided in Section 16.

16. *Range of Fluctuation.*—Some fluctuation in the results of the rattler test, both on account of variations in the bricks and in the machine used in testing, are unavoidable, and a reasonable allowance for such fluctuations should be made, wherever the standard may be fixed.

In any lot of paving brick, if the loss on a test computed upon its initial weight exceeds the standard loss by more than 2 per cent, then the portion of the lot represented by that test shall at once be resampled and three more tests executed upon it, and if any of these three tests shall again exceed by more than 2 per cent the required standard, then that portion of the lot shall be rejected.

If in any lot of brick, two or more tests exceed the permissible maximum, then the buyer may at his option reject the entire lot, even though the average of all the tests executed may be within the required limits.

17. *Fixing of Standards.*—The percentage of loss which may be taken as the standard, will not be fixed in these specifications, and shall remain within the province of the contracting parties. For the information of the public, the following scale of average losses is given, representing what may be expected of tests executed under the foregoing specifications:

	General Average Loss, Per Cent.	Maximum Permissible Loss, Per Cent.
For bricks suitable for heavy traffic.....	22	24
For bricks suitable for medium traffic.....	24	26
For bricks suitable for light traffic.....	26	28

Which of these grades should be specified in any given district and for any given purpose is a matter wholly within the province of the buyer, and should be governed by the kind and amount of traffic to be carried, and the quality of paving bricks available.

18. *Culling and Retesting.*—Where, under Sections 15 and 16, a lot or portion of a lot of bricks is rejected, either by reason of failure to show a low enough average test or because of tests above the permissible maximum, the buyer may at his option permit the seller to regrade the rejected brick, separating out that portion which he considers at fault and retaining that which he considers good. When the regrading is complete, the good portion shall be then resampled and retested, under the original conditions, and if it fails again either in average or in permissible maximum, then the buyer may definitely and finally reject the entire lot or portion under test.

19. *Payment of Cost of Testing.*—Unless otherwise specified, the cost of testing the material as delivered or prepared for delivery, up to the prescribed number of tests for valid acceptance or rejection of the lot, shall be paid by the buyer. (See also Section 23.) The cost of testing extra samples made necessary by the failure of the whole lot or any portion of it, shall be paid by the seller, whether the material is finally accepted or rejected.

## II. VISUAL INSPECTION

It shall be the right of the buyer to inspect the bricks, subsequent to their delivery at the place of use, and prior to or during laying, to cull out and reject upon the following grounds:

20. All bricks which are broken in two or chipped in such a manner that neither wearing surface remains substantially intact, or that the lower or bearing surface is reduced in area by more than one-fifth. Where bricks are rejected upon this ground, it shall be the duty of the purchaser to use them so far as practicable in obtaining the necessary half-bricks for breaking courses and making closures, instead of breaking otherwise whole and sound bricks for this purpose.

21. All bricks which are cracked in such a degree as to produce defects such as are defined in Section 20, either from shocks received in shipment and handling, or from defective conditions of manufacture, especially in drying, burning or cooling, unless such cracks are plainly superficial and not such as to preceptibly weaken the resistance of the brick to its conditions of use.

22. All bricks which are so off-size, or so misshapen, bent, twisted or kiln-marked, that they will not form a proper surface as defined by the paving specifications, or align with other bricks without making joints other than those permitted in the paving specifications.

23. All bricks which are obviously too soft and too poorly vitrified to endure street wear. When any disagreement arises between buyer and seller under this item, it shall be the right of the buyer to make two or more rattler tests of the brick which he wishes to exclude, as provided in Section 2, and if in either or both tests, the bricks fall beyond the maximum rattler losses permitted under the specifications, then all bricks having the same objectionable appearance may be excluded, and the seller shall pay for the cost of the test. But if under such procedure, the bricks which have been tested as objectionable, shall pass the rattler test, both tests falling within the permitted maximum, then the buyer cannot exclude the class of material represented by this test and he shall pay for the cost of the test.

24. All bricks which differ so markedly in color from the type or average of the shipment, as to make the resultant pavement checkered or disagreeably mottled in appearance. This Section shall not be held to apply to the normal variations in color which may occur in the product of one plant among bricks which will meet the rattler test as referred to in Sections 15, 16, and 17, but shall apply only to differences of color which imply differences in the material of which the bricks are made, or extreme differences in manufacture.

## APPENDIX B

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### ABRAMS' FINENESS MODULUS METHOD FOR PROPORTIONING CONCRETE

*Acknowledgment.*—Since writing Chapters XIII and XIV, the authors have been accorded the privilege of publishing the following brief abstract of certain results of tests made under the direction of Professor D. A. Abrams, of Lewis Institute, Chicago.\* Inasmuch as these data establish definite relations between the consistency of the mix, the sieve analysis of the aggregate and the strength of the concrete, and, since they also furnish a much more scientific basis for proportioning concrete and mortar, we feel very grateful to Professor Abrams for this privilege. We are especially indebted for the figures, tables and the notes which he has furnished for this brief discussion.

*The Effect of Proportion of Mixing Water on the Strength of Concrete.*—As the result of about a dozen extensive series of tests in which the water content, the size of the aggregate and the age of the test pieces were variables, Professor Abrams concludes that the use of the correct amount of mixing water is fundamental. Fig. 1 shows the relation between compressive strength of concrete and the ratio of the volume of water to the volume of cement in the mix for one of these series of tests. It should be noted that the maximum size of aggregate in the mixes ranged from that which passed a No. 14-mesh to that which passed a 2-in. opening. The variation in the richness of the mixes is noted in the legend on the figure. Only mixes of plastic or wetter consistency form a basis for this figure. If the data for dry mixes had been plotted, there would have resulted a series of hooked curves extending downward to the left from the main curve in Fig. 1. A study of this figure shows very clearly how greatly the strength is decreased by using an excess of mixing water.†

*The Fineness Modulus* may be defined as the sum of the percentages in the sieve analysis divided by 100, using Tyler standard sieves and expressing

\* The laws developed are based upon approximately 50,000 tests of mortars and concretes. The Structural Materials Research Laboratory at Lewis Institute is run under the co-operation of the Portland Cement Association and Lewis Institute, Professor Abrams being in charge.

† For further information concerning this series of tests see *Engineering News-Record*, May 2, 1918; for additional data on other series, see *Canadian Engineer*, Vol. 35, p. 73, 103, 132.

the analysis in terms of the total quantity of material coarser than each sieve. The sieves used by Professor Abrams are the 100, 48, 28, 14, 8, 4,  $\frac{3}{4}$ ,  $\frac{3}{8}$ ,  $1\frac{1}{2}$  in., made of square-mesh wire cloth. (See Art. 462.) This fineness modulus has been shown by Professor Abrams to be a remarkable index of the strength-making quality of any aggregate. Also it furnishes a means of proportioning two or more given aggregates so that the resultant mixture will make the best concrete which can be obtained from the given materials.

If the fineness modulus method of proportioning is compared with the Fuller method, which also makes use of mechanical analysis (see Art. 483), it will be found that the Fuller method ensures mixes of high fineness modulus but it requires the use of aggregate graded in a fixed manner.

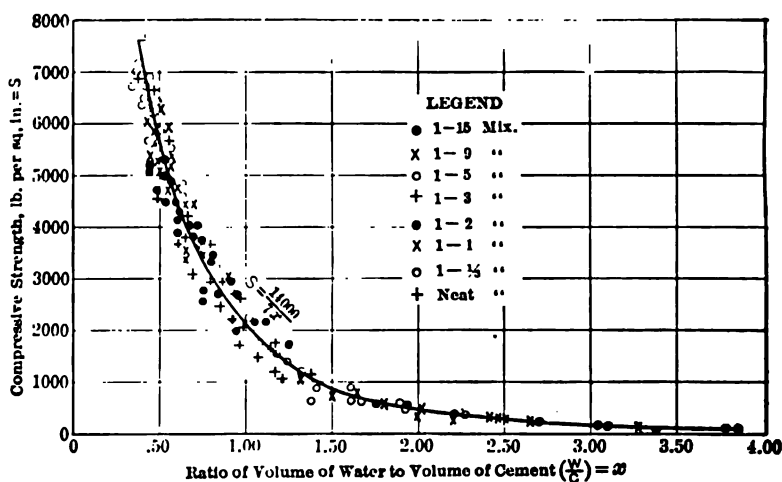


FIG. 1.—Effect of Ratio  $\frac{\text{Volume of Water}}{\text{Volume of Cement}}$  on Strength of Concrete.

Consistencies normal or wetter; proportions by volume, 1 cu. ft. of cement = 94 lb. Data represent 1600 tests on 6 × 12 in. cylinders. (Abrams in *Concrete Highway Magazine*, May, 1918.)

In many instances such gradation is prohibited by the expense involved in screening or by the grading of the natural aggregates. The Abrams method, however, enables one to use to best advantage any given set of materials. If, however, it is necessary to screen the aggregate, in order to secure the very best strength, then the Abrams method will in general require less screening than the Fuller method.

Table 1 shows the method of computing the fineness modulus for several different grades of sand and gravel. For material of one size standard sand for example the fineness modulus may be computed from the equation:  $m = 7.94 + 3.32 \log_{10} d$ . Here  $m$  = fineness modulus and  $d$  = average diameter of particles in inches. This equation holds for any single size of aggregate provided the sieves above mentioned are used.

**TABLE 1.—METHOD OF CALCULATING FINENESS MODULUS OF AGGREGATES. (ABRAMS)**

The fineness modulus of an aggregate is the sum of the percentages given by the sieve analysis divided by 100.

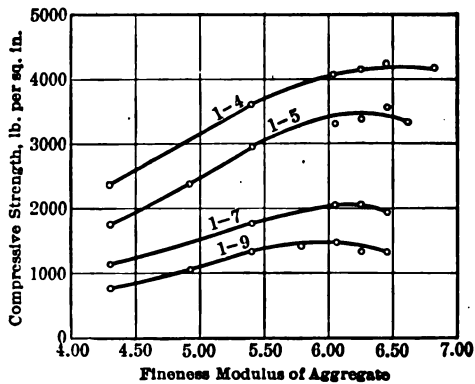
The sieves used are commonly known as the Tyler standard sieves. It will be noted that the clear opening of each sieve is just double that of the preceding one.

The sieve analysis may be expressed in terms of volume or weight.

Sieve Size.	SIZE OF SQUARE OPENING.		SIEVE ANALYSIS OF AGGREGATES, PERCENTAGE COARSER THAN A GIVEN SIEVE.						
			Sand.			Pebbles.			Concrete Aggregate (G)*
	In.	Mm.	Fine (A)	Medium (B)	Coarse (C)	Fine (D)	Medium (E)	Coarse (F)	
100	.0058	.147	82	91	97	100	100	100	93
48	.0116	.295	52	70	81	100	100	100	92
28	.0232	.590	20	46	63	100	100	100	86
14	.043	1.79	0	24	44	100	100	100	81
8	.093	2.36	0	10	25	100	100	100	78
4	.185	2.70	0	0	0	86	95	100	71
2	.37	9.4	0	0	0	51	66	86	48
1	.75	18.8	0	0	0	9	25	50	20
1½	1.5	37.6	0	0	0	0	0	0	0
Fineness modulus.....			1.54	2.41	3.10	6.46	6.86	7.33	5.74

\* Concrete aggregate G is made up of 25 per cent of sand B mixed with 75 per cent of pebbles E. Equivalent gradings would be secured by mixing 33 per cent of sand B with 67 per cent of coarse pebbles F; 28 per cent of sand A with 72 per cent of pebbles F, etc.

*Relation of Fineness Modulus to Strength of Concrete.*—Fig. 2 shows the relation between the fineness modulus of the aggregate and the strength of concrete of varying proportions made from different aggregates. It will be noted that the peak in the curve occurs for higher values of the fineness modulus with rich mixes than with lean mixes. If a curve showing the relation between compressive strength and fineness modulus were plotted on Fig. 2 for a very rich mix, like a 1 : 1 mortar, it would be nearly horizontal, showing that there is little advantage in careful proportioning of



**FIG. 2.—Effect of Fineness Modulus of Aggregate on Strength of Concrete. (ABRAMS.)**

Each point represents 5 tests of 6×12-in. cylinders; consistency normal (=1.00); age=28 days; aggregate was sand and pebbles graded up to 1½ in. in diameter.

rich mixes. The curves

in Fig. 2 are representative of the relation which exists for concrete mixes ordinarily used and for aggregate of a given maximum size.

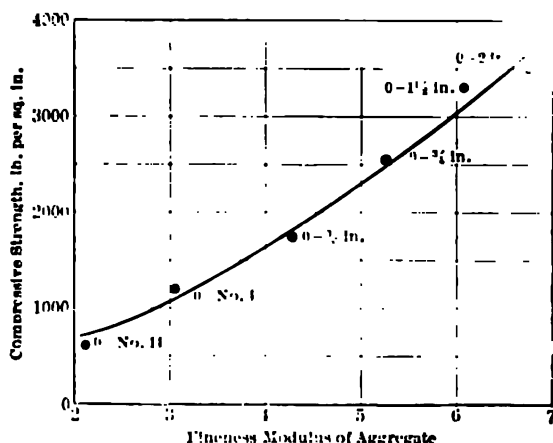


FIG. 3. Effect of Gradation of Aggregate on Strength of Concrete (Abrams.)

Each point represents 5 tests on 6" x 12-in. cylinders of 1:5 proportions at 28 days. Aggregate was sand and pebbles. Consistency was normal.

Fig. 3 illustrates the relation between the compressive strength of concrete and the fineness modulus of the aggregate when the maximum size of aggregate is variable. The proportion of cement to aggregate was in all cases 1 : 5.

Inspection of Figs. 2 and 3 indicates that within the range of sizes and proportions commonly used the strength of concrete increases directly with the fineness modulus.

**Maximum Permissible Values for the Fineness Modulus.**—Experience in using the fineness modulus as a means of proportioning concrete has shown that it is not practicable to increase the fineness modulus beyond certain limits, these limits depending upon the character of the aggregate, the maximum size of the coarse aggregate, the gradation of the fine aggregate, the richness of the mixture, and the use for the concrete. The desirable maximum limits for various mixes and sizes of aggregate are tabulated in Table 2.

TABLE 2.—MAXIMUM PERMISSIBLE VALUES OF FINENESS MODULUS OF AGGREGATES. (ABRAMS)

For mixes other than those given in the table, use the values for the next leaner mix.

For maximum sizes of aggregate other than those given in the table, use the values for the next smaller size.

This table is based on the requirements for *sand-and-pebble or gravel aggregate* composed of approximately spherical particles, in ordinary uses of concrete in reinforced concrete structures. For other materials and in other classes of work the maximum permissible values of fineness modulus for an aggregate of a given size is subject to the following corrections:

(1) If *crushed stone or slag* is used as coarse aggregate, *reduce* values in table by 0.25. For crushed material consisting of unusually flat or elongated particles, *reduce* values by 0.40.

(2) For *pebbles* consisting of *flat particles*, *reduce* values by 0.25.

(3) If *stone screenings* are used as fine aggregate, *reduce* values by 0.25.

(4) For the top course in *concrete roads*, or other work requiring a smooth finish, *reduce* the values by 0.25. If finishing is done by *mechanical means*, this reduction need not be made.

(5) In work of *massive proportions*, such that the smallest dimension is larger than 10 times the maximum size of the coarse aggregate, *additions may be made to the values* in the table as follows: for  $\frac{1}{2}$ -in. aggregate, 0.10; for  $1\frac{1}{2}$ -in., 0.20; for 3-in., 0.30; for 6-in., 0.40.

Sands with fineness modulus lower than 1.50 are undesirable as fine aggregate in ordinary concrete mixes. Natural sands of such fineness are seldom found.

*Sand or screenings* used for fine aggregate in concrete must not have a higher fineness modulus than that permitted for mortars of the same mix. Mortar mixes are covered by the table and by (3) above.

*Crushed stone* mixed with both finer sand and coarser pebbles requires no reduction in fineness modulus provided the quantity of crushed stone is less than 30 per cent of the total volume of the aggregate.

Size of Aggregate.	PROPORTIONS (BY VOLUME). AGGREGATE: CEMENT.								
	1	2	3	4	5	6	7	9	12
Sieve No.									
Mortars									
0-28	2.25	2.00	1.85	1.70	1.60	1.50	1.40	1.30	1.20
0-14	3.00	2.70	2.50	2.30	2.15	2.05	1.95	1.85	1.80
0- 8	3.80	3.40	3.10	2.90	2.75	2.65	2.55	2.45	2.40
0- 4	4.75	4.20	3.90	3.60	3.45	3.30	3.20	3.05	2.95
In.									
0-3*	5.25	4.60	4.30	4.00	3.80	3.65	3.55	3.45	3.35
0- $\frac{3}{4}$	5.60	5.05	4.70	4.40	4.20	4.05	3.95	3.85	3.80
0- $\frac{1}{2}$ *	6.05	5.45	5.10	4.80	4.60	4.45	4.35	4.25	4.20
0- $\frac{1}{4}$	6.50	5.90	5.50	5.20	5.00	4.85	4.75	4.65	4.60
0-1*	6.90	6.30	5.90	5.60	5.40	5.25	5.15	5.00	5.00
0-1 $\frac{1}{2}$	7.35	6.70	6.30	6.00	5.80	5.65	5.55	5.40	5.35
0-2.1*	7.75	7.10	6.70	6.40	6.20	6.05	5.95	5.80	5.75
0-3	8.20	7.55	7.15	6.85	6.60	6.50	6.40	6.25	6.20
0-4 $\frac{1}{2}$ *	8.65	7.95	7.55	7.25	7.00	6.90	6.80	6.65	6.60
0-6	9.10	8.40	8.00	7.65	7.45	7.30	7.20	7.05	7.00

\* Half sieves; not used in computing fineness modulus.

*Method of Determining the Amount of Mixing Water.*—In view of the extremely important influence of the consistency of the mix on the strength of the resulting concrete it is very necessary to carefully proportion the mixing water. From many thousands of tests Professor Abrams has evolved the following formula:

$$x = R \left[ \frac{3}{2} p + \left( \frac{.30}{1.26^m} + a - c \right) n \right].$$

$x$  = ratio of the volume of water to the volume of cement in mix.

$p$  = the amount of water required for normal consistency in the standard test for cement, expressed as a ratio of the weight of the cement.

$m$  = fineness modulus of the mixed aggregate.

$a$  = the ratio of the volume of water absorbed by the dry aggregate to the volume of the aggregate, after immersion in water for three hours. (An average value for crushed limestone and gravel is 0.02. Porous sandstones may absorb 0.08.)

$c$  = the ratio of the volume of moisture in the aggregate to volume of aggregate ( $c=0$  for air-dry aggregate).

$n$  = ratio of volume of aggregate to volume of cement in mix.

$R$  = relative consistency of concrete. ( $R=1.00$ —a normal consistency obtains when the quantity of mixing water is such that a 6×12-in. cylinder made in a smooth metal mold by puddling with a small rod will just stand, if the form is removed immediately after molding by a steady upward pull. A relative consistency of 1.20 means the use of 20 per cent more water than required for normal consistency.)

This equation gives the proper amount of water to make concrete of the same plasticity or workability, regardless of the proportion of cement or size and grading of aggregate, provided the relative consistency  $R$  is made constant.

The amounts of water required for concrete or mortar of normal consistency made from cement requiring an average amount of water for normal consistency and an aggregate in semi-dry condition have been tabulated in Table 3. For any other relative consistency multiply all values in the table by the desired factor.

For common mixes and ranges in aggregate gradation, the following equation is sufficiently exact and more simple to use:

$$x = R \left[ \frac{3}{2} p + \left( 0.22 - \frac{m}{42} + a - c \right) n \right].$$



TABLE 3.—PROPORTIONATE AMOUNTS OF WATER REQUIRED TO SECURE CONCRETE OR MORTAR OF NORMAL CONSISTENCY

Computed from formula  $x = R \left[ \frac{3}{2}p + \left( \frac{0.30}{1.26^m} + a - c \right)n \right]$ .  $R=1$ ,  $p=0.22$ ,  $(a-c)=0.01$ .

Aggregate Cement (by Volume)	IN TERMS OF VOLUME OF CEMENT FOR FINENESS MODULI OF						GALLONS OF WATER PER SACK OF CEMENT FOR FINENESS MODULI OF					
	2	3	4	5	6	7	2	3	4	5	6	7
1	.53	.49	.46	.43	.41	.40	4.0	3.7	3.5	3.2	3.1	3.0
2	.73	.65	.59	.54	.50	.47	5.5	4.9	4.4	4.0	3.8	3.6
3	.93	.81	.72	.64	.58	.54	7.0	6.1	5.4	4.8	4.4	4.1
4	1.12	.97	.85	.75	.67	.61	8.4	7.3	6.4	5.6	5.0	4.6
5	1.32	1.13	.98	.85	.75	.68	9.9	8.5	7.4	6.4	5.6	5.1
6	1.52	1.29	1.10	.96	.84	.75	11.4	9.7	8.3	7.2	6.3	5.6
7	1.72	1.45	1.23	1.06	.92	.82	12.9	10.9	9.2	8.0	6.9	6.2
8	1.92	1.61	1.33	1.17	1.01	.89	14.4	12.1	10.2	8.8	7.6	6.7
9	2.12	1.77	1.49	1.27	1.09	.96	15.9	13.3	11.2	9.5	8.2	7.2
10	2.32	1.93	1.62	1.38	1.18	1.03	17.4	14.5	12.1	10.3	8.9	7.7

*Outline of Method for Designing Concrete Mixes.*—The following steps have been suggested by Prof. Abrams as a guide in designing concrete mixes.

"1. Knowing the approximate compressive strength required of the concrete and the maximum size of aggregate which may be used in the work, estimate the dryest 'relative consistency' which may be used in the mix. (The mix is expressed as 1 volume of cement to a given number of volumes of aggregate; that is, the combined fine and coarse aggregate.)

"2. Make sieve analysis of fine and coarse aggregates, using Tyler standard sieves of the following sizes: 100, 48, 28, 14, 8, 4,  $\frac{3}{4}$ ,  $\frac{3}{8}$ , and  $1\frac{1}{2}$  in. Express sieve analysis in terms of percentages of material, by weight (or separate volumes), *coarser than* each of the standard sieves.

"3. Compute fineness modulus of each aggregate by adding the percentages found in (2).

"4. Determine 'maximum size' of aggregate by applying the following rules: If more than 20 per cent of aggregate is coarser than any sieve the maximum size shall be taken as the next larger sieve in the regular series; if between 11 and 20 per cent is coarser than any sieve, maximum size shall be the next larger half sieve;\* if less than 10 per cent is coarser than any sieve that sieve shall be considered the maximum size.

"5. From Table 2 determine the maximum value of fineness modulus which may be used for the mix, kind and size of aggregate under consideration.

\* Half sieves are listed with an \* in Table 2.

" 6. Compute the percentages of fine and coarse aggregates required to produce the fineness modulus values desired for the aggregate mixture by applying the formula:

$$y = \left( \frac{A - B}{A - C} \right) 100,$$

where  $y$  = percentage of fine aggregate in total mixture;

$A$  = fineness modulus of coarse aggregate;

$B$  = fineness modulus of final aggregate mixture;

$C$  = fineness modulus of fine aggregate.

" 7. With the estimated mix, fineness modulus and consistency enter Fig. 4 and determine the strength of concrete produced by the combination. If the strength shown by the diagram is not that required, the necessary readjustment may be made by changing the mix, consistency, or size and grading of the aggregates.

" **IMPORTANT NOTE.** It must be understood that the values in Fig. 4 were determined from compression tests of 6 by 12-in. cylinders stored for twenty-eight days in a damp place. The values obtained on the work will depend on such factors as the consistency of the concrete, quality of the cement, methods of mixing, handling, placing the concrete, etc., and on age and curing conditions.

" Strength values higher than that given for relative consistency of 1.10 should seldom be considered in designing, since it is only in unusual cases that a consistency dryer than this can be economically used. For wetter concrete much lower strengths must be considered."

The method of using the chart in Fig. 4 is indicated by the dotted lines. For example, suppose the strength of a 1 : 3 mixture of mortar made from a sand having a fineness modulus of 3.00 is required. Draw a line from the point marked 1 : 3, on the line of mixes at the left of the diagram, through the point marked 3 on the line designated " fineness modulus of aggregate." Mark the intersection of this line with the " reference line for consistency," and read the strength. In this case it is 3000 lb. per square inch. If a consistency other than normal is to be used, the strength may be found by drawing a horizontal line through the intersection of the first line and the reference line for consistency. In this problem the strength for the relative consistency of 1.20 is approximately 2200 lb. per square inch.

By the chart it is also possible to compare the strengths of different mixes made from aggregates varying in fineness modulus. For example, the 1 : 3 mortar mentioned above is somewhat less strong than a 1 : 5 mix made from aggregate having a fineness modulus of 5.7. (See lower dotted line in chart.)

Again it is possible to determine the proper proportions when bank-run gravel is substituted for a mix containing fine and coarse aggregate and

having a known strength. For example, suppose a specification requires 1 : 2 : 4 concrete with the strength of 2000 lb. per square inch and it is proposed that a bank-run gravel having a fineness modulus of 4 be sub-

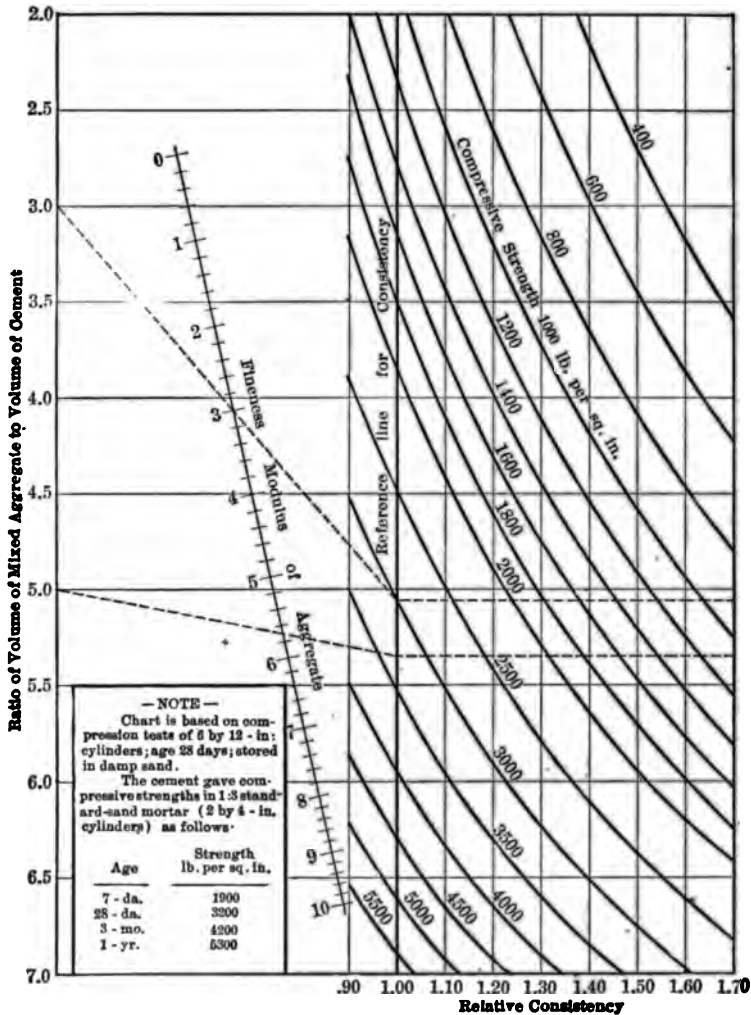


FIG. 4.—Abrams' Chart for Designing Concrete Mixes

Diagram shows relations between proportions, fineness modulus, consistency, and strength.

stituted for this mix, the "relative consistency" being 1.20. By inspection of the chart we find that the proportions should be approximately 1 : 3.9.

For a typical problem in mix design, suppose that a mix with a strength of 3000 lb. per square inch at one month is wanted for reinforced concrete columns. The available aggregates are sand *B* and pebbles *D* of Table 1. Assume proportions of 1 : 4.5 and a relative consistency, 1.20. Since only 9 per cent of the pebbles is coarser than a  $\frac{3}{4}$ -in. sieve, we enter Table 2 on the line marked 0  $\frac{3}{4}$ -in. and find that the maximum permissible fineness modulus for the above mix is 5.10. Then  $y = \left( \frac{6.46 - 5.10}{6.46 - 2.41} \right) 100 = 34$  per cent. Consequently we should use 34 per cent of sand *B* and 66 per cent of pebbles *D* in making the aggregate mixture. Entering Fig. 4 with a 1 : 4.5 mix, 5.10 fineness modulus, and a "relative consistency" of 1.20 we find that the strength of this mix is only 2300 lb. per square inch. A 1 : 3.5 mix which allows a fineness modulus of 5.35 for the aggregate, gives the required strength, the proportion of sand *B* and pebbles *D* in the aggregate being 27 and 73, respectively. If the reinforcement in the columns is widely spaced and it is possible to thoroughly puddle the columns, the relative consistency might be reduced to 1.10. With this consistency a 1 : 4 mix containing an aggregate with 31 per cent of sand *B* and 69 per cent of pebbles *D* has sufficient strength. By interpolating in Table 3 we find that the amount of water required for normal consistency by the last mix, having a fineness modulus of 5.2, is 5.5 gal. per sack of cement. For a "relative consistency" of 1.1, 6.05 gal. per sack of cement is required.

# APPENDIX C

## LIST OF STANDARDS

ADOPTED BY THE

## AMERICAN SOCIETY FOR TESTING MATERIALS

These Standards are all copyrighted in the name of the American Society for Testing Materials. Permission to reprint any of these Standards can be obtained only from the Executive Committee on application to the Secretary-Treasurer.<sup>1</sup>

The designations A 1, A 2, etc., of the Standards are fixed; the final numbers 14, 12, etc., indicate the year of adoption as standard, or in the case of revision, the year of last revision. The serial designations of Standards which have for any reason been discontinued are permanently dropped. In ordering Standards, the complete serial designations should be stated, thus: A 1-14, A 2-12, etc.

### A. FERROUS METALS

#### Steel

(See also Wrought Iron: A 56-18)

#### STANDARD SPECIFICATIONS

##### Steel Rails and Accessories

- A 1-14. For Carbon-Steel Rails.
- A 2-12. For Open-hearth Steel Girder and High Tee Rails.
- A 3-14. For Low-Carbon-Steel Splice Bars.
- A 4-14. For Medium-Carbon-Steel Splice Bars.
- A 5-14. For High-Carbon-Steel Splice Bars.
- A 6-14. For Extra-High-Carbon-Steel Splice Bars.
- A 49-15. For Quenched High-Carbon-Steel Splice Bars.
- A 50-16. For Quenched Carbon-Steel Track Bolts.
- A 51-16. For Quenched Alloy-Steel Track Bolts.
- A 65-18. For Steel Track Spikes.
- A 66-18. For Steel Screw Spikes.

##### Structural Steels

- A 7-16. For Structural Steel for Bridges.
- A 8-16. For Structural Nickel Steel.
- A 9-16. For Structural Steel for Buildings.
- A 10-16. For Structural Steel for Locomotives.
- A 11-16. For Structural Steel for Cars.
- A 12-16. For Structural Steel for Ships.
- A 13-14. For Rivet Steel for Ships.

##### Spring Steel and Springs

- A 14-16. For Carbon-Steel Bars for Railway Springs.
- A 68-18. For Carbon-Steel Bars for Railway Springs with Special Silicon Requirements.
- A 58-16. For Carbon-Steel Bars for Vehicle and Automobile Springs.
- A 59-16. For Silico-Manganese-Steel Bars for Automobile and Railway Springs.
- A 60-16. For Chrome-Vanadium Steel Bars for Automobile and Railway Springs.
- A 61-16. For Helical Steel Springs for Railways.
- A 62-16. For Elliptical Steel Springs for Railways.
- A 69-18. For Elliptical Steel Springs for Automobiles.

##### Reinforcement Bars

- A 15-14. For Billet-Steel Concrete Reinforcement Bars.
- A 16-14. For Rail-Steel Concrete Reinforcement Bars.

##### Steel Blooms, Forgings and Axles

- A 17-18. For Carbon-Steel and Alloy-Steel Blooms, Billets and Slabs for Forgings.
- A 18-18. For Carbon-Steel and Alloy-Steel Forgings.
- A 19-18. For Quenched-and-Tempered Carbon-Steel Axles, Shafts, and Other Forgings for Locomotives and Cars.
- A 63-18. For Quenched-and-Tempered Alloy-Steel Axles, Shafts, and Other Forgings for Locomotives and Cars.
- A 20-16. For Carbon-Steel Forgings for Locomotives.
- A 21-18. For Carbon-Steel Car and Tender Axles.
- A 22-16. For Cold-Rolled Steel Axles.

##### Steel Wheels and Tires

- A 57-16. For Wrought Solid Carbon-Steel Wheels for Steam Railway Service.
- A 25-16. For Wrought Solid Carbon-Steel Wheels for Electric Railway Service.
- A 26-16. For Steel Tires.

##### Steel Castings

- A 27-16. For Steel Castings.

##### Steel Tubes and Pipe

- A 28-18. For Lap-Welded and Seamless Steel Boiler Tubes for Locomotives.
- A 52-18. For Lap-Welded and Seamless Steel and Wrought-Iron Boiler Tubes for Stationary Service.
- A 53-18. For Welded Steel Pipe.

##### Automobile Steels (see also Spring Steel)

- A 29-18. For Automobile Carbon and Alloy Steels.

<sup>1</sup>The Secretary-Treasurer has headquarters at the University of Pennsylvania, Philadelphia, Pa.

**Boiler Steels**

- A 30 18. For Boiler and Firebox Steel for Locomotives.  
A 31 14. For Boiler Rivet Steel.

**Cold-Drawn Steels]**

- A 32 14. For Cold-Drawn Bessemer Steel Automatic Screw Stock.  
A 54-15. For Cold-Drawn Open-hearth Steel Automatic Screw Stock.

**STANDARD TESTS**

- A 34-18. For Magnetic Properties of Iron and Steel.

**STANDARD METHODS**

- A 33-14. For Chemical Analysis of Plain Carbon Steel.  
A 55-15. For Chemical Analysis of Alloy Steels.

**RECOMMENDED PRACTICE**

- A 35 11. For Annealing of Miscellaneous Rolled and Forged Carbon-Steel Objects.  
A 36-14. For Annealing of Carbon-Steel Castings.  
A 37-14. For Heat Treatment of Case-Hardened Carbon-Steel Objects.

**Wrought Iron**

(See also Steel: A 52-18)

**STANDARD SPECIFICATIONS**

- A 38 18. For Lap-Welded Charcoal-Iron Boiler Tubes for Locomotives.  
A 72 18. For Welded Wrought-Iron Pipe.  
A 39 18. For Staybolt Iron.  
A 40-18. For Engine-Bolt Iron.

- A 41 18. For Refined Wrought-Iron Bars.  
A 42-18. For Wrought-Iron Plates.  
A 56 18. For Iron and Steel Chain.  
A 73-18. For Wrought-Iron Rolled or Forged Blooms and Forgings for Locomotives and Cars.

**Pig Iron, Cast Iron, and Finished Castings****STANDARD SPECIFICATIONS**

- A 43 09. For Foundry Pig Iron.  
A 44 04. For Cast-Iron Pipe and Special Castings.  
A 74 18. For Cast-Iron Soil Pipe and Fittings.  
A 45 14. For Cast-Iron Locomotive Cylinders.  
A 46 05. For Cast-Iron Car Wheels.

- A 47 15. For Malleable-Iron Castings.  
A 48-18. For Gray-Iron Castings.

**STANDARD METHODS**

- A 64-16. For Sampling and Analysis of Pig and Cast Iron.

**B. NON-FERROUS METALS****STANDARD SPECIFICATIONS****Ingot Copper**

- B 4 13. For Lake Copper Wire Bars, Cakes, Slabs, Billets, Ingots, and Ingot Bars.  
B 5 13. For Electrolytic Copper Wire Bars, Cakes, Slabs, Billets, Ingots, and Ingot Bars.

**Spelter**

- B 6 18. For Spelter.

**Bronze**

- B 7 14. For Manganese-Bronze Ingots for Sand Castings.  
B 10 18. For the Alloy: Copper, 88 per cent; Tin, 10 per cent; Zinc, 2 per cent.

**Wire and Cable.**

- B 1 15. For Hard-Drawn Copper Wire.

- B 2-15. For Medium Hard-Drawn Copper Wire.

- B 3-15. For Soft or Annealed Copper Wire.  
B 8-16. For Bare Concentric-Lay Copper Cable: Hard, Medium-Hard, or Soft.  
B 9-16. For High-Strength Bronze Trolley Wire, Round and Grooved: 40 and 65-per-cent Conductivity.

**Copper and Brass Plates, Tubes, etc.**

- B 11-18. For Copper Plates for Locomotive Fireboxes.  
B 12-18. For Copper Bars for Locomotive Staybolts.  
B 13-18. For Seamless Copper Boiler Tubes.  
B 14-18. For Seamless Brass Boiler Tubes.  
B 15-18. For Brass Forging Rod.  
B 16-18. For Free-Cutting Brass Rod for Use in Screw Machines.

**C. CEMENT, LIME, GYPSUM, AND CLAY PRODUCTS****STANDARD SPECIFICATIONS**

- C 9 17. Standard Specifications and Tests for Portland Cement.  
C 10 09. For Natural Cement.  
C 4 16. For Drain Tile.  
C 5 15. For Quicklime.  
C 6 15. For Hydrated Lime.

- C 7-15. For Paving Brick.  
C 19-18. For Fire Tests of Materials and Construction.

**STANDARD DEFINITIONS**

- C 8-15. Of Terms Relating to Sewer Pipe.

**D. MISCELLANEOUS MATERIALS.****STANDARD SPECIFICATIONS****Preservative Coatings**

- D 1-15. For Purity of Raw Linseed Oil from North American Seed.
- D 11-15. For Purity of Boiled Linseed Oil from North American Seed.
- D 12-16. For Purity of Raw Tung Oil.
- D 13-15. For Turpentine.

**Coke**

- D 17-16. For Foundry Coke.

**Timber**

- D 10-15. For Yellow-Pine Bridge and Trestle Timbers.

**Rubber**

- D 14-15. For 2½-in. Cotton Rubber-Lined Fire Hose for Private Department Use.
- D 26-18. For 2½, 3, and 3½-in. Double-Jacketed Cotton Rubber-Lined Fire Hose for Public Fire Department Use.
- D 46-18. For Air-Line Hose for Pneumatic Tools.

**STANDARD TESTS****Preservative Coatings**

- D 28-17. For Paint Thinners Other than Turpentine.
- D 29-17. For Shellac.

**Lubricants**

- D 47-18. For Lubricants.

**Road Materials**

- D 2-08. For Abrasion of Road Material.
- D 3-18. For Toughness of Rock.
- D 30-18. For Determination of Apparent Specific Gravity of Coarse Aggregates.
- D 4-11. For Soluble Bitumen.
- D 5-16. For Penetration of Bituminous Materials.
- D 6-16. For Loss on Heating of Oil and Asphaltic Compounds.

**STANDARD METHODS****Preservative Coatings**

- D 34-17. For Routine Analysis of White Pigments.
- D 49-18. For Routing Analysis of Dry Red Lead.
- D 50-18. For Routing Analysis of Yellow, Orange, Red and Brown Pigments containing Iron and Manganese.

**Road Materials**

- D 7-18. For Making a Mechanical Analysis of Sand or Other Fine Highway Material, except for Fine Aggregates Used in Cement Concrete.
- D 18-16. For Making a Mechanical Analysis of Broken Stone or Broken Slag, except for Aggregate Used in Cement Concrete.
- D 19-16. For Making a Mechanical Analysis of Mixtures of Sand or Other Fine Material with Broken Stone or Broken Slag, except for Aggregates Used in Cement Concrete.
- D 35-18. Form of Specifications for Certain Commercial Grades of Broken Stone.
- D 20-18. For Distillation of Bituminous Materials Suitable for Road Treatment.

**Coal and Coke**

- D 21-16. For Sampling of Coal.
- D 22-16. For Laboratory Sampling and Analysis of Coal.
- D 37-18. For Laboratory Sampling and Analysis of Coke.

**Timber Preservative**

- D 38-18. For Sampling and Analysis of Creosote Oil.

**Rubber**

- D 15-15. For Testing of Cotton Rubber-Lined Hose.

**STANDARD DEFINITIONS**

- D 16-15. Of Terms Relating to Paint Specifications.
- D 8-18. Of Terms Relating to Materials for Roads and Pavements.
- D 9 13. Of Terms Relating to Structural Timber.

**E. MISCELLANEOUS SUBJECTS****STANDARD METHODS**

- E 1-18. For Testing.
  - I. For Tension Tests of Metals.
  - II. For Compression Tests of Metals.

- III. For Transverse Tests of Metals.
- IV. For Brinell Hardness Tests of Metals.
- V. For Metallographic Tests of Metals.

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## PUBLISHED BY THE

### AMERICAN SOCIETY FOR TESTING MATERIALS

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#### A. FERROUS METALS

##### Steel

TENTATIVE SPECIFICATIONS	
A 67-18 T. For Steel Tie Plates.	A 71-17 T. For Carbon Tool Steel.
A 70-18 T. For Boiler and Firebox Steel for Stationary Service.	A 76 18 T. For Low-Carbon-Steel Track Bolts.
	A 77 18 T. For Electric Cast Steel Anchor Chain.

##### Cast Iron

##### TENTATIVE SPECIFICATIONS

A 75 18 T. For Malleable Castings.

#### B. NON-FERROUS METALS

TENTATIVE SPECIFICATIONS	
B 17-18 T. For Non-Ferrous Alloys for Railway Equipment in Ingots, Castings, and Finished Car and Tender Bearings.	B 24 18 T. Aluminum Ingots for Remelting and for Rolling.
B 19 18 T. For Cartridge Brass.	B 25 18 T. For Aluminum Sheet.
B 20 18 T. For Cartridge Brass Licks.	B 26 18 T. For Light Aluminum Casting Alloys.
B 21-18 T. For Naval Brass Rods for Structural Purposes.	
B 22 18 T. For Bronze Bearing Metals for Turntables and Movable Railroad Bridges.	
B 23-18 T. For White Metal Bearing Alloys (known commercially as "Babbitt Metal").	

##### TENTATIVE METHODS

B 18 17 T. For Chemical Analysis of Alloys of Lead, Tin, Antimony and Copper.
B 27 18 T. For Chemical Analysis of Manganese Bronze.
B 28 18 T. For Chemical Analysis of Gun Metal.

#### C. CEMENT, LIME, GYPSUM, AND CLAY PRODUCTS.

TENTATIVE SPECIFICATIONS	TENTATIVE METHODS
C 9 16 T. Specifications and Tests for Compressive Strength of Portland-Cement Mortars. <sup>1</sup>	C 18 18 T. For Ultimate Chemical Analysis of Refractory Materials.
C 13 18 T. For Clay Sewer Pipe.	C 20-18 T. For Determination of Porosity and Permanent Volume Changes in Refractory Materials.
C 14 18 T. For Cement-Concrete Sewer Pipe.	
C 15 17 T. For Required Safe Crushing Strengths of Sewer Pipe to Carry Loads from Ditch Filling.	
C 6 17 T. For Mason's Hydrated Lime. <sup>2</sup>	
TENTATIVE TESTS	TENTATIVE DEFINITIONS
C 16 18 T. For Refractory Materials under Load at High Temperatures.	C 11 16 T. Of Terms Relating to the Gypsum Industry.
C 17 17 T. For Slagging Action of Refractory Materials.	
	TENTATIVE RECOMMENDED PRACTICE
	C 12 17 T. For Laying Sewer Pipe.

<sup>1</sup>To be added, when adopted, to the Standard Specifications and Tests for Portland Cement (Serial Designation: C 9 17).

<sup>2</sup>To replace, when adopted, the Standard Specifications for Hydrated Lime (Serial Designation: C 6-15).



**D. MISCELLANEOUS MATERIALS****TENTATIVE SPECIFICATIONS****Preservative Coatings**

- D 51-18 T. For Foots Permissible in Properly Clarified Pure Raw Linseed Oil from North American Seed.

**Timber**

- D 23-16 T. For Selected Structural Douglas Fir Bridge and Trestle Timbers.  
 D 24-15 T. For Southern Yellow-Pine Timber to be Creosoted.  
 D 25-15 T. For Southern Yellow-Pine Piles and Poles to be Creosoted.  
 D 52-18 T. For Wooden Paving Blocks for Exposed Pavements.

**Waterproofing**

- D 40-17 T. For Asphalt for Use in Damp-proofing and Water-proofing.  
 D 41-17 T. For Primer for Use with Asphalt for Use in Damp-proofing and Water-proofing.  
 D 42-17 T. For Coal-Tar Pitch for Use in Damp-proofing and Water-proofing.  
 D 43-17 T. For Creosote Oil for Priming Coat with Coal-Tar Pitch for Use in Damp-proofing and Water-proofing.

**Shipping Containers**

- D 44-17 T. For Canned Foods Boxes, Nailed and Lock-Corner Construction.  
 D 45-17 T. For Canned Foods Boxes, Wirebound Construction.

**Rubber Products**

- D 27-16 T. For Insulated Wire and Cable: 30-per-cent Hevea Rubber.  
 D 53-18 T. For Rubber Belting for Power Transmission.  
 D 54-18 T. For Steam Hose.

**TENTATIVE TESTS**

- D 28-18 T. For Flash Point of Paint Thinners Other than Turpentine.<sup>1</sup>  
 D 55-18 T. For Determination of Apparent Specific Gravity of Sand, Stone, and Slag Screenings, and Other Fine Non-Bituminous Highway Materials.  
 D 48-17 T. For Molded Insulating Materials.

**TENTATIVE METHODS**

- D 36-16 T. For Determination of Softening Point of Bituminous Materials Other than Tar Products.  
 D 39-18 T. For Testing Cotton Fabrics.

**E. MISCELLANEOUS SUBJECTS****TENTATIVE DEFINITIONS**

- E 2-18 T. Definitions and Rules Governing the Preparation of Micrographs of Metals and Alloys.

To be added, when adopted, to the Standard Tests for Paint Thinners other than Turpentine (Serial Designation: D 28-17).

1

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